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The Coupled Trickling Filter-Activated Sludge Process: Design and Performance

Brown and Caldwell, Walnut Creek, CA

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A case history report was prepared on the upgrading of the Livermore, California, Water Reclamation Plant from a conventional trickling filter plant with tertiary oxidation ponds to a coupled trickling filter-activated sludge plant producing a nitrified effluent low in BOD_5, suspended solids, and coliform organisms.

The report covers planning, design, construction, startup, and operation and performance of the upgraded Livermore plant. Capital costs and operation and maintenance expenses are also given. Data and information from Livermore were used in conjunction with data from other coupled trickling filter-activated sludge plants to develop general design considerations for carrying out similar upgradings elsewhere.

Over 7 yr of records from Livermore show that the coupled trickling filter-activated sludge process is extremely stable and reliable. Effluent BOD_5 and suspended solids concentrations of 10 to 20 mg/l can be obtained, along with ammonia nitrogen concentrations less than 1 mg/l. Monthly median total coliform concentrations of 2.1 MPN/100 ml were consistently achieved at Livermore using high chlorine dosages and a chlorine contact tank with good hydraulic characteristics.

The coupled trickling filter-activated sludge process is particularly adaptable to existing conventional trickling filter plants where stringent, new discharge requirements have been imposed and where existing structures and equipment are in good condition and can be used in an upgraded facility.

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THE COUPLED TRICKLING FILTER-ACTIVATED SLUDGE PROCESS: DESIGN AND PERFORMANCE

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This case history report illustrates the combined use of careful long-range planning and technical ingenuity in upgrading an existing conventional trickling filter plant to increase capacity and meet more stringent effluent discharge requirements. No existing facilities were abandoned in the upgrading process. The procedures described herein are recommended for consideration by other engineers faced with similar upgrading situations.

Francis T. Mayo, Director
Municipal Environmental Research
Laboratory

ABSTRACT

A case history report was prepared on the upgrading of the Livermore, California, Water Reclamation Plant from a conventional trickling filter plant with tertiary oxidation ponds to a coupled trickling filter-activated sludge plant producing a nitrified effluent low in BOD_5, suspended solids, and coliform organisms.

The report covers planning, design, construction, startup, and operation and performance of the upgraded Livermore plant. Capital costs and operation and maintenance expenses are also given. Data and information from Livermore were used in conjunction with data from other coupled trickling filter-activated sludge plants to develop general design considerations for carrying out similar upgradings elsewhere.

Over 7 yr of operating records from Livermore show that the coupled trickling filter-activated sludge process is extremely stable and reliable. Effluent BOD_5 and suspended solids concentrations of 10 to 20 mg/l can be obtained, along with ammonia nitrogen concentrations less than 1 mg/l. Monthly median total coliform concentrations of 2.1 MPN/100 ml were consistently achieved at Livermore using high chlorine dosages and a chlorine contact tank with good hydraulic characteristics.

The coupled trickling filter-activated sludge process is particularly adaptable to existing conventional trickling filter plants where stringent, new discharge requirements have been imposed and where existing structures and equipment are in good condition and can be used in an upgraded facility. Principal design considerations are aeration tank size, aeration air supply, flexibility, reliability, and efficient use of existing facilities.

This report was submitted in fulfillment of Contract No. 68-03-2175 by Brown and Caldwell, Consulting Engineers, under the sponsorship of the U. S. Environmental Protection Agency. Plant operating and performance data are included in this case history report for the period of January 1968 through December 1974.

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Other individuals at Brown and Caldwell who participated in design and construction of the Livermore Water Reclamation Plant and who contributed to this study are David H. Caldwell, David L. Eisenhauer, Nestor D. Vivado, and Warren R. Uhte. We are particularly indebted to the City of Livermore and the staff of the Livermore Water Reclamation Plant for information and assistance provided during the course of the study.

SECTION 1

INTRODUCTION

Current emphasis in the United States on reducing the level of contaminants in wastewater effluents has resulted in many communities finding that their wastewater treatment facilities, although well designed, constructed, and operated, cannot meet the stringent, new discharge requirements imposed by regulatory agencies. It is sometimes more economical to abandon recently constructed facilities in favor of completely new plants designed to provide better treatment, but often it is possible to incorporate all or part of existing plants into upgraded facilities capable of producing the high quality effluent required today.

Trickling filters using rock media have long been a common form of biological treatment in the United States. Used mostly in small and moderate size communities, they are capable of providing good removal of organic contaminants and are noted as being stable, reliable, and economical. By themselves, however, rock trickling filters (preceded by primary treatment) cannot provide the high effluent quality often required today. Discharge requirements often specify much lower levels of BOD_5, suspended solids, and coliform organisms than in the past, and ammonia, nutrient, and toxicity removal may also be required.

For a community utilizing trickling filtration, the most economical way to meet more stringent discharge requirements may be to incorporate the existing filters in an upgraded facility. The reliability, resistance to upsets, and efficient organic removal provided by trickling filters can be used to advantage in developing a treatment scheme to consistently and economically produce a high quality effluent.

The City of Livermore, California (Figures 1 and 2), followed such a course in upgrading its wastewater treatment facility to meet new disinfection requirements imposed in 1965 by the California Regional Water Pollution Control Board, San Francisco Bay Region. Livermore is located approximately 70 km (45 mi) east of San Francisco in an area where rapid urbanization over the last 30 yr has led to serious water and wastewater management problems. The Livermore-Amador Valley contains the city of Livermore, the nearby University of California Lawrence Livermore Laboratory, the city of Pleasanton,

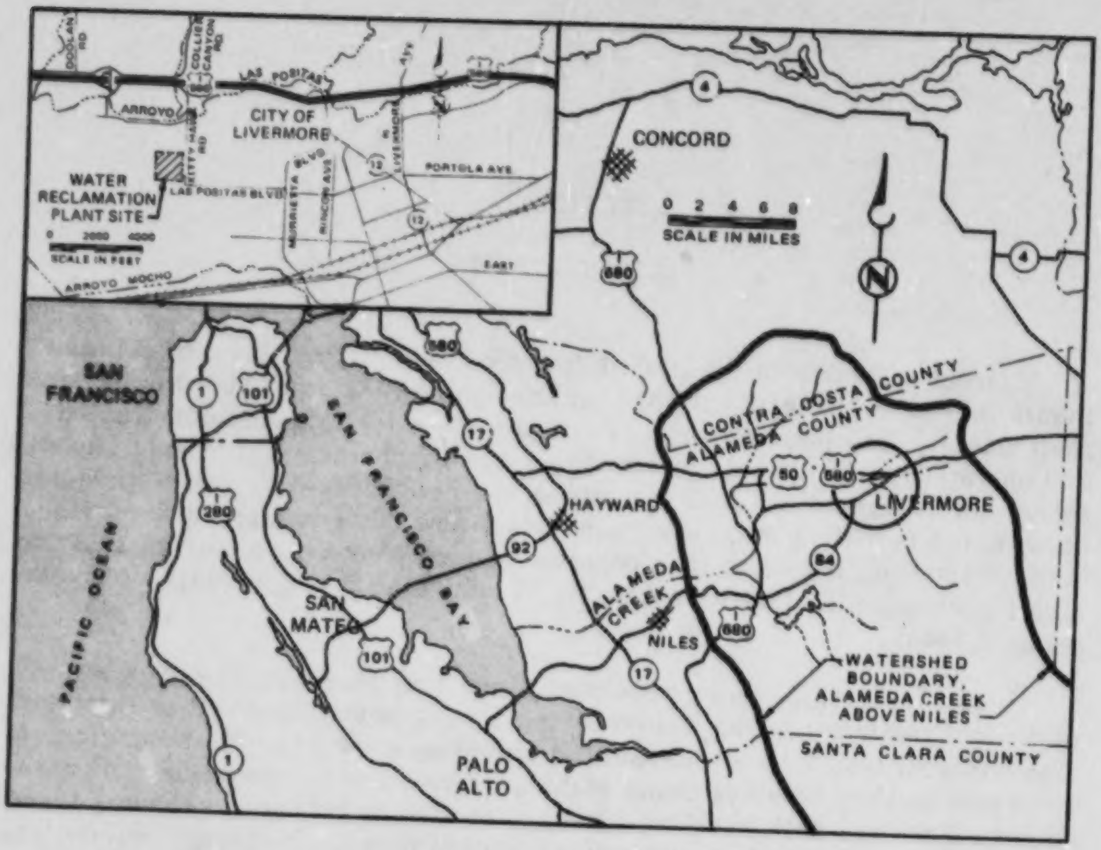

DOOLAN RD
COLLIER CANYON RD
LAS POSITAS
ARROYO
CITY OF LIVERMORE
WATER RECLAMATION PLANT SITE
LAS POSITAS BLVD
MURRIETA BLVD
RINCON AVE
LIVERMORE AVE
PORTOLA AVE
12
0 2000 4000
SCALE IN FEET
ARROYO MOCHO
EAST
CONCORD
0 2 4 6 8
SCALE IN MILES
N
4
680
580
SAN FRANCISCO
1
101
SAN FRANCISCO BAY
17
CONTRA COSTA COUNTY
ALAMEDA COUNTY
50
580
LIVERMORE
HAYWARD
280
92
PACIFIC OCEAN
84
ALAMEDA CREEK
SAN MATEO
NILES
680
101
17
WATERSHED BOUNDARY, ALAMEDA CREEK ABOVE NILES
SANTA CLARA COUNTY
PALO ALTO

Figure 1. Location of Livermore Water Reclamation Plant.

and the unincorporated communities of Dublin and San Ramon. The valley is part of the Alameda Creek watershed above the Niles district of the City of Fremont, California, and has an area of approximately 1,700 km^2 (660 mi^2). Although Alameda Creek is tributary to San Francisco Bay, much of the concern over water quality centers on the creek itself and on the Niles cone, a groundwater basin which is recharged by waters flowing from Alameda Creek. Increased use of local surface water and groundwaters in the Livermore-Amador Valley, coupled with increased wastewater flows, has resulted in salinity increases in downstream waters and concern for possible health problems resulting from recreational use of local streams.

As part of a multifaceted effort to improve water quality in the area, the Regional Water Pollution Control Board adopted on July 15, 1965, stringent new discharge requirements for the City of Livermore's wastewater treatment

Figure 2. Aerial view of Livermore - Amador Valley. Rapid urbanization in recent years has led to serious water quality problems.

plant (Resolution No. 683, see Appendix B). These discharge requirements included a provision that the 5-day median total coliform concentration not exceed 5.0 MPN/100 ml. To meet this requirement, the existing Livermore plant, during expansion from an average dry weather flow (ADWF) capacity of 0.11 m^3/sec (2.5 mgd) to 0.22 m^3/sec (5.0 mgd), was upgraded from a conventional trickling filter system to a coupled trickling filter-activated sludge plant capable of producing a nitrified effluent which can be economically disinfected to the required level.

This report has been prepared to describe the upgrading of the Livermore plant, including planning, design, construction, startup, and operation and performance over a 7-yr period from January 1968 through December 1974.

Construction and operating costs are also presented, along with general design considerations for upgrading trickling filter plants to the coupled trickling filter-activated sludge mode.

OBJECTIVES AND SCOPE

This review of the Livermore plant upgrading has been undertaken to make available information which may be useful to communities and engineering consultants who are faced with situations similar to that which occurred at Livermore. Because the upgraded Livermore Water Reclamation Plant has been in operation since 1967, a substantial quantity of operating data are available for study so that a detailed evaluation of plant performance can be developed.

Specific objectives are identified as follows:

1. Present information on conversion to the upgraded facility. This includes preliminary planning, detailed design, construction, and capital costs. Special emphasis is given to incorporation of nitrification in the process flow sheet and the relationship between the added activated sludge unit and other unit processes.
2. Review operation of the upgraded plant. Difficulties encountered in startup and operation are discussed, along with operational techniques developed to counter such problems. Performance data for the startup period and the 7-yr operational period that followed are presented. Particular attention is given to comparing performance with design objectives. Operation and maintenance costs are also documented.
3. Develop general design considerations for upgrading conventional trickling filter plants to the coupled trickling filter-activated sludge operational mode. Experience from the Livermore plant is emphasized, but information from other planned or constructed plants with similar flow sheets is also utilized. Special emphasis is given to process design for nitrification in the activated sludge unit.

OUTLINE OF REPORT

This report has been organized to present first a chronological history of the Livermore plant and then to discuss specific aspects of plant operation and performance before setting out general design considerations. Sections 4 through 8 review the background, design, construction, startup, and operation of the Livermore Water Reclamation Plant. The specific task of comparing performance with design objectives is also covered in Section 8. Section 9 reviews in detail both capital and operating costs for the Livermore plant. Information developed in Sections 4 through 9 is then augmented by information from other sources for presentation in Section 10, General Design Considerations.

SECTION 2

CONCLUSIONS

Use of the coupled trickling filter-activated sludge process for biological wastewater treatment at Livermore, California, has resulted in reliable, consistent production of a wastewater effluent low in BOD_5, suspended solids, ammonia nitrogen, and coliform organisms. Constructed on the site of a conventional rock media trickling filter plant, the upgraded plant (Figure 3) utilized many portions of the original facility. The excellent performance of the plant since it was placed in operation in 1967 suggests that the coupled trickling filter-activated sludge process should be considered by communities which operate existing trickling filter plants that cannot meet new, more stringent discharge requirements imposed by regulatory agencies.

The trickling filter acts as a roughing filter ahead of the activated sludge process to remove carbonaceous oxygen demand. Nitrification in the aeration tank removes ammonia nitrogen from the waste stream and further reduces the carbonaceous BOD. The principal advantage of the roughing filter is the protection it provides for the more easily upset activated sludge process and for the nitrifying organisms which are susceptible to toxic materials. Acting as a buffer, it ensures a stability of operation that is not associated with the activated sludge process alone. Where existing facilities are in good condition and space is available, upgrading to the coupled trickling filter-activated sludge process can be undertaken to provide nitrification, either where ammonia nitrogen removal is desired or ahead of subsequent denitrification for nitrogen removal.

Specific conclusions can be drawn from the operation of the Livermore Water Reclamation Plant, from recent designs of other coupled trickling filter-activated sludge plants, and from the nitrification process kinetics of the trickling filter-activated sludge process:

1. Secondary effluent BOD_5 and suspended solids concentrations of 10 to 20 mg/l can be obtained with the coupled trickling filter-activated sludge process, along with ammonia nitrogen concentrations of less than 1 mg/l (nitrification at Livermore may have been limited by an inadequate air supply). Monthly median total coliform organism concentrations of 2.1 MPN/100 ml

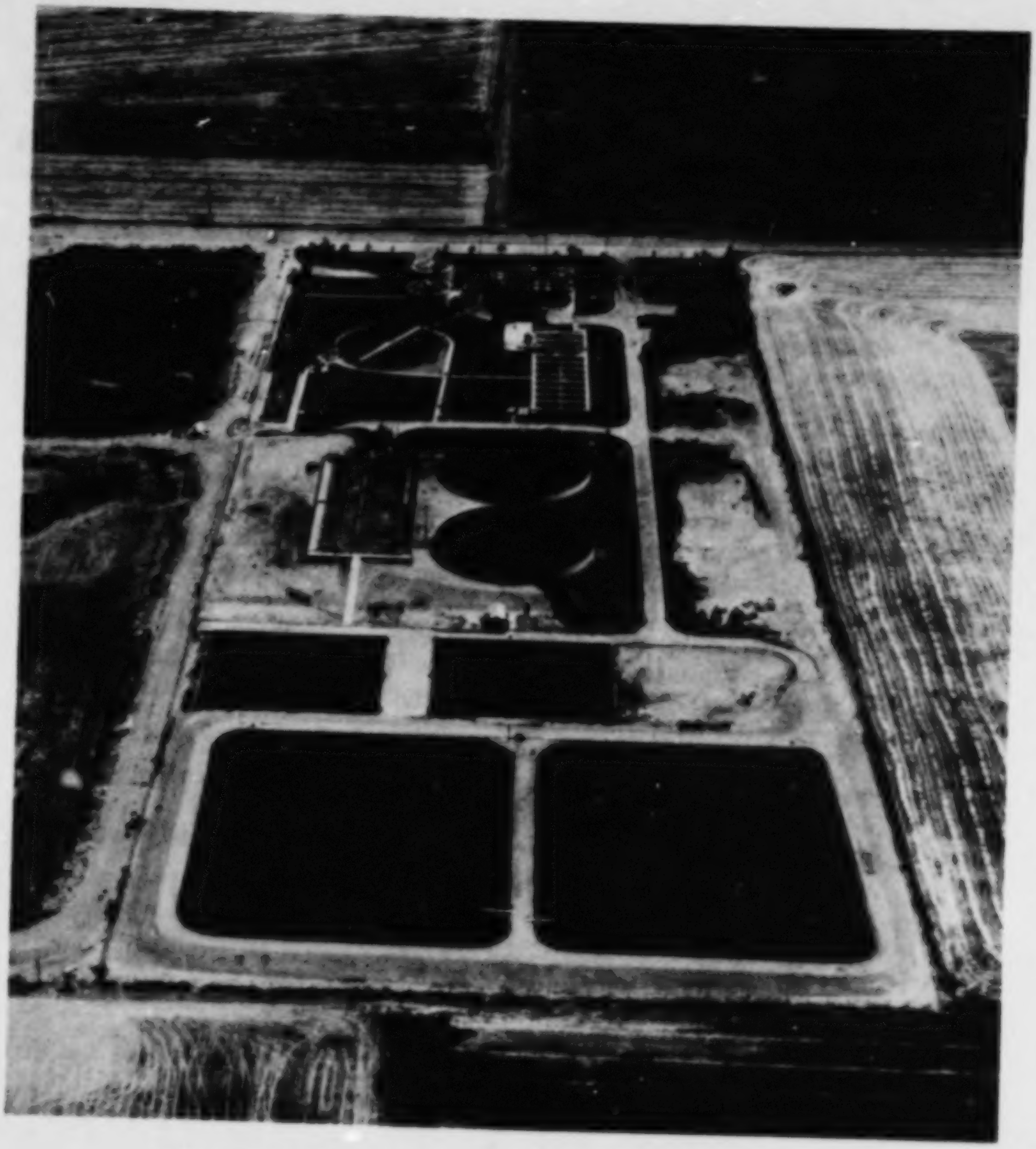


Figure 3. Aerial view of upgraded Livermore Water Reclamation Plant. Plant was upgraded by placing an activated sludge unit between rock media trickling filters and secondary clarifier in flow diagram.

and 7-day medians of 5.0 MPN/100 ml can also be consistently obtained if the coupled process is combined with efficient disinfection.

2. In designing solids handling facilities for an upgraded plant, account must be taken of the increased solids production associated with increased BOD removal and the associated difficulty in dewatering the waste activated sludge produced.

3. Use of emergency holding basins can ensure that discharge of inadequately treated effluent will not occur during periods when process units are shut down. Existing oxidation ponds can often be modified for use as holding basins.

4. As existing secondary clarifiers will normally need to be modified substantially to accommodate the activated sludge process, it may be economical to convert them to other uses in the upgraded plant and to construct new secondary clarifiers. Possible conversions include primary sedimentation, sludge thickening, and chlorine contact.

5. The most efficient use of existing facilities will differ for each individual case, and considerable ingenuity may be required to determine the best approach.

6. The characteristics of unsettled trickling filter effluent have not been studied extensively; this hinders the application of rational design methods to the coupled trickling filter-activated sludge process.

SECTION 3

RECOMMENDATIONS

This study has shown that the coupled trickling filter-activated sludge process as used at Livermore, California, is an extremely stable wastewater treatment process capable of producing a nitrified effluent low in BOD_5 and suspended solids. Principal recommendations for future study pertain to the necessity of developing information for use in process design of the coupled trickling filter-activated sludge system. Specifically, the characteristics of unsettled trickling filter effluent must be better defined in order to fully develop a rational design procedure for the process.

Measurements of biochemical oxygen demand on unsettled trickling filter effluent will reflect both unoxidized waste material passing through the trickling filter and biological solids sloughed from the filter media. Data from Livermore indicate that a reasonable estimate is that 50 percent of the BOD_5 measured on an unsettled sample is due to sloughed material, but further evaluation is needed.

Further work is also needed to predict the quantity of organisms entering the aeration tank from the trickling filter and whether they can be treated in the same manner as organisms which grow in the aeration tank. In this report a net yield coefficient is applied to the BOD_5 removed in the roughing filter to obtain a value for cell concentration entering the aeration tank. Values for this net yield coefficient should be determined as well as decay rate coefficients for cells in the aeration tank.

SECTION 4

BACKGROUND

The City of Livermore, California, was first settled in 1869 when the Southern Pacific Railroad constructed a depot in the area. Named after one of its early settlers, Robert Livermore, the City was incorporated 7 yr later in 1876. The City was mostly a farming community in its early days with grain and cattle its main commodities, and even today agricultural activity remains an important part of the area's economy. Prior to 1927 sewage treatment was effected in a community septic tank with discharge to land. In that year an Imhoff tank and percolation beds were constructed west of the City The plant was expanded in 1942 by the addition of a comminutor and a primary clarifier* operating in parallel with the Imhoff tank.

After World War II, the City and the Livermore-Amador Valley experienced the rapid growth associated with the San Francisco Bay Area as a whole. This resulted in overloading of the treatment plant with consequent odor problems, principally from sludge disposal. Additionally, wells near the disposal area were found to contain 40 to 60 mg/l of nitrate, posing a possible public health hazard. Nominal capacity of the treatment plant was 0.025 m^3/sec (0.58 mgd), and development in the Livermore area was being hindered by lack of adequate wastewater treatment and disposal facilities.

In 1956,the City began development of a plan for construction of new treatment and disposal facilities and improvements to the existing sewerage system. The resulting facility, designed by Brown and Caldwell, Consulting Engineers, was a 0.11-m^3/sec (2.5-mgd) plant located approximately 3 km (2 mi) west of the city center on existing farmlands. Plant units consisted of preliminary treatment, primary clarification, trickling filtration, secondary clarification (in clarifiers intended for conversion to primary sedimentation tanks during future expansion), and tertiary oxidation ponds. The layout of the plant as well as the sizing of many units was based on expected ultimate expansion to an average dry weather flow of 0.44 m^3/sec (10 mgd) with hydraulic capacity for 1.6 m^3/sec (36 mgd). The flow diagram and layout for the original plant are shown in Figures 4 and 5, respectively. Design data are presented in Table 1.

---

*The terms "clarifier" and "sedimentation tank" are used synonymously in this report.

Design BOD_5 and suspended solids removals were 85 percent for primary plus secondary treatment; these removals corresponded to secondary effluent concentrations of 50 mg/l for BOD_5 and 35 mg/l for suspended solids. Plant performance data for 1964 are shown in Table 2. Discharge requirements for the original plant are presented in Appendix A (Regional Water Pollution Control Board Resolution No. 239, adopted March 21, 1957).

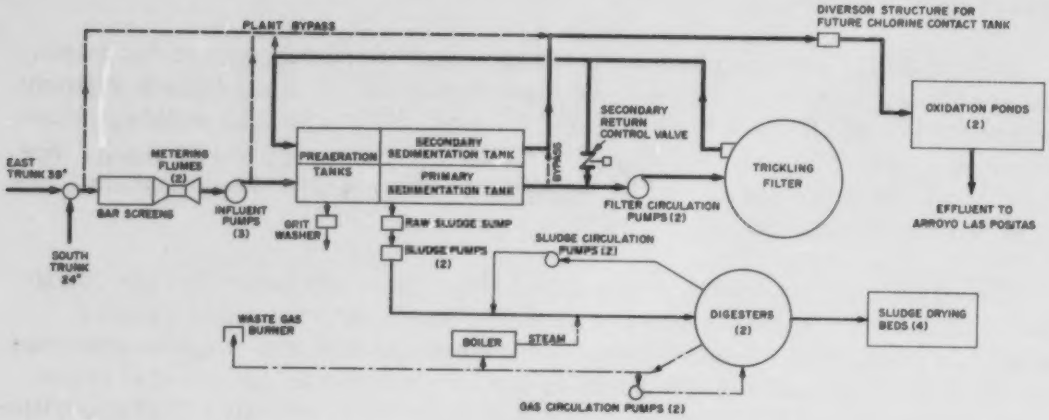
The plant was put into operation in 1957. Although under design conditions effluent was to be discharged from the ponds to Arroyo Las Positas (which is tributary to Alameda Creek), high percolation and evaporation rates from the oxidation ponds prevented any surface discharge for several years. In late 1962 and in 1963 increased flows coupled with clogging of the infiltrative surfaces in the ponds resulted in periodic discharges to the sometimes dry stream. Complaints about color and detergent foaming in the receiving waters occurred immediately. At this time a chlorine contact tank was added to the flow diagram ahead of the oxidation ponds to provide disinfection.

A fortuitous coincidence was the cessation of nonbiodegradable detergent (alkylbenzenesulfonate or ABS) manufacturing in the United States. Substitution of linear alkylate sulfonate (LAS) in home laundry detergents resulted in elimination of the foaming problem within a few months.

In the period 1963-65, the City's population grew rapidly at an annual rate of 15-20 percent. In April 1964,a feasibility study prepared for the City by Brown and Caldwell recommended expansion of the existing plant to 0.22 m^3/sec (5.0 mgd) capacity utilizing essentially the same treatment scheme.

However, because of the surface water quality problems which had recently occurred, subsequent discharge requirements issued in July 1965 by the San Francisco Bay Regional Water Pollution Control Board (Appendix B) called for effluent BOD_5 and suspended solids concentrations of 20 mg/l; grease, 5 mg/l; settleable solids, 0.5 ml/l; and total coliforms, 5.0 MPN/100 ml, based on a 5-day median. (The disinfection requirement was subsequently changed in 1971 to 2.2 MPN/100 ml, based on a 7-day median.)

The disinfection requirement was unusually stringent and for effluent from a conventional secondary treatment plant would have required the addition of sufficient chlorine to react with all the ammonia present in the wastewater in order to produce free chlorine residuals. The result would have been high operating costs and possible adverse side effects such as odors and toxic effects in receiving waters.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DIVERSON STRUCTURE FOR FUTURE CHLORINE CONTACT TANK
OXIDATION PONDS (2)
SECONDARY RETURN CONTROL VALVE
EAST TRUNK 30"
METERING FLUMES (2)
PREAERATION TANKS
SECONDARY SEDIMENTATION TANK
PRIMARY SEDIMENTATION TANK
BYPASS
TRICKLING FILTER
BAR SCREENS
INFLUENT PUMPS (3)
GRIT WASHER
RAW SLUDGE SUMP
FILTER CIRCULATION PUMPS (2)
EFFLUENT TO ARROYO LAS POSITAS
SOUTH TRUNK 24"
SLUDGE PUMPS (2)
SLUDGE CIRCULATION PUMPS (2)
WASTE GAS BURNER
BOILER
STEAM
DIGESTERS (2)
SLUDGE DRYING BEDS (4)
GAS CIRCULATION PUMPS (2)

Figure 4. Flow diagram for original Livermore plant.

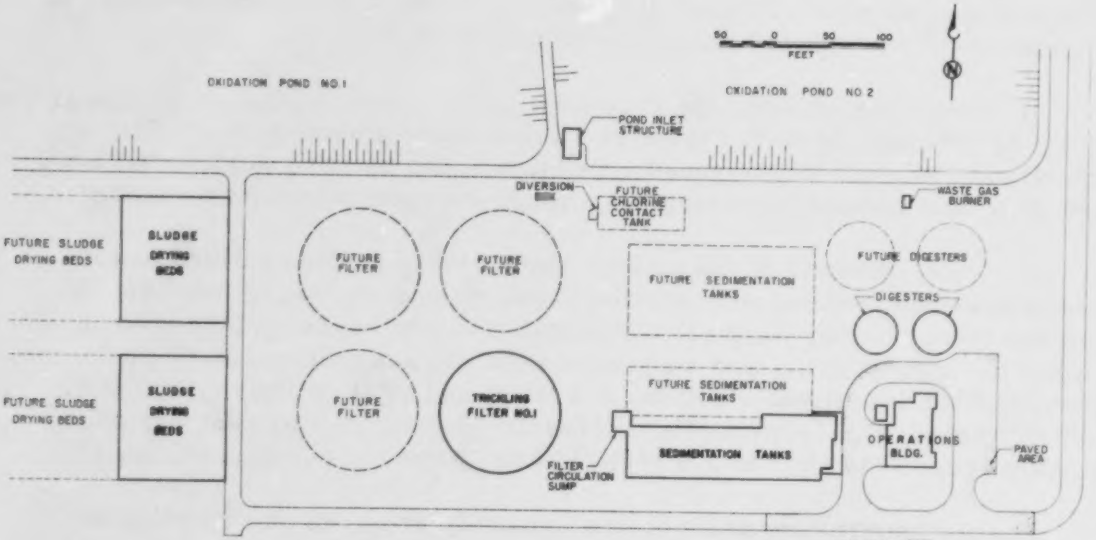

50
0
50
100
FEET
N
OXIDATION POND NO.1
OXIDATION POND NO 2
POND INLET STRUCTURE
DIVERSION
FUTURE CHLORINE CONTACT TANK
WASTE GAS BURNER
FUTURE SLUDGE DRYING BEDS
SLUDGE DRYING BEDS
FUTURE FILTER
FUTURE FILTER
FUTURE SEDIMENTATION TANKS
FUTURE DIGESTERS
DIGESTERS
FUTURE SLUDGE DRYING BEDS
SLUDGE DRYING BEDS
FUTURE FILTER
TRICKLING FILTER NO.1
FUTURE SEDIMENTATION TANKS
SEDIMENTATION TANKS
OPERATIONS BLDG.
PAVED AREA
FILTER CIRCULATION SUMP
WELL SLAB

Figure 5. Layout of original Livermore plant.

TABLE 1. DESIGN DATA FOR ORIGINAL LIVERMORE PLANT

Design Factor	Value
Flow, mgd[a]	
Initial design, average dry weather	2.5
Initial design, maximum dry weather	4.5
Initial design, peak storm rate	10.0
Future design, average dry weather	10.0
Future design, maximum dry weather	19.5
Future design, peak storm rate	36.0
Design Loadings	
Initial design, population equivalent	28,900
Future design, population equivalent	129,000
Suspended solids, lb/day[b]	
Per capita	0.15
Initial design	4,300
Future design	19,000
BOD$_5$, lb/day[b]	
Per capita	0.20
Initial design	5,800
Future design	26,000
Incoming Sewer	
Diameter, in.[c]	42
Capacity, mgd[a]	38
Bar Screens	
Number	
Initial design	1
Future design	2
Width, ft[d, 1]	4
Water depth, (maximum) ft[d]	3.5
Water area, ft^2[e]	14
Velocity through bar screen, ft/sec[d]	
Present minimum (one channel)	0.9
Initial design, ADWF (one channel)	1.5
Initial peak storm rate (one channel)	2.5
Future design, ADWF (two channels)	1.3
Future peak storm rate (two channels)	2.0
Parshall Flumes	
Number	2
Throat width, in.[c]	15
Discharge head, ft[d]	
Present minimum (one channel)	0.25
Initial design, ADWF (one channel)	0.84
Initial peak storm rate (one channel)	2.1
Future design, ADWF (two channels)	1.3
Future peak storm rate (two channels)	3.1
Raw Sewage Pumps	
Number	3
Capacity each, mgd[a]	12
Static lift, ft[d]	11
Preaeration Tanks	
Number	
Initial design [2]	2
Future design	4
Width, ft[d]	19
Length, ft[d]	38
Average water depth, ft[d]	11.7
Detention time, hr (one tank)	0.6
Air supplied per tank, ft^3/min[f, 3]	200
Air supplied, ft^3/gal[g]	0.12
Maximum hydraulic capacity, mgd[a]	10
Primary Sedimentation Tanks	
Number	
Initial design [4]	2
Future design	4
Width, ft[d]	19
Length, ft[d]	124
Average water depth, ft[d]	9
Effluent weir length per tank, ft[d]	164
Detention time, hr	1.5
Mean forward velocity, ft/min[d]	1.4
Overflow rate at ADWF, gpd/ft^2[h]	1,050
Maximum hydraulic capacity, mgd[a]	10
Assumed removals, percent	
Suspended solids	60
BOD$_5$	35
Secondary Sedimentation Tanks	
Number	
Initial design [4]	0
Future design	4
Width, ft[d]	19
Length, ft[d]	160
Average water depth, ft[d]	9
Detention time, hr	2.0
Trickling Filter Circulation Pumps	
Number	
Initial design	2
Future design	6
Maximum capacity per unit, mgd[a]	
No. 1 (No's 3, 4, 6 future)	7.6
No. 2 (No. 5 future)	3.8
Average static lift, ft[d]	12.5
Trickling Filters	
Number	
Initial design	1
Future design	4
Inside diameter, ft[d]	110
Average depth of filter media, ft[d]	4.25
Size of filter media, in.[c]	2 to 4
Net area of filter surface, ft^2[e]	0.218
Volume, 1,000 ft^3/filter[f]	0.92
Circulation ratio to ADWF	1.5 to 3.0
Loading	
Rate per filter, mgd[a]	7.5
Rate per unit surface area, gpd/ft^2[h]	34.5
BOD$_5$, lb/1,000 ft^3/day[i]	110
Assumed removal, filter plus secondary sedimentation, percent	
Suspended solids	60
BOD$_5$	75
Oxidation Ponds	
Total area, acres[j]	
Initial design	37
Future design	150
Average water depth, ft[d]	6
Detention time, days (neglecting evaporation and seepage)	
At design ADWF (2.5 mgd[a])	30
At future ADWF (10.0 mgd[a])	30
BOD$_5$ loading, lb/acre/day[k]	
Initial design	28
Future design	28

(continued on next page)

TABLE 1. (continued)

Design Factor	Value
Digestion Tanks	
Number	
Initial design	2
Future design	4
Inside diameter, ft[d]	
Initial two tanks	35
Future two tanks	55
Side water depth [5], ft[d]	27.5
Volume, 1,000 ft^3 [f]	
Initial tanks	53
Future tanks	157
Total future volume, four tanks	210
Loading, 1,000 lb dry solids/day[b]	
Initial design	3.8
Future design	16
Unit loading, lb dry solids/ft^3/day [l]	
Initial design	0.07
Future design	0.08
Assumed solids reduction, percent	45
Assumed gas production at 6.0 ft^3/lb settleable solids[m]	
Initial design, 1,000 ft^3/day[f]	23
Future design, 1,000 ft^3/day[f]	96
Dry sludge produced, 1,000 lb/day[b]	
Initial design	2.1
Future design	8.8
Assumed moisture of digested sludge, percent	95
Volume of digested sludge produced, 1,000 ft^3/day[f]	
Initial design	0.7
Future design	2.8

Design Factor	Value
Sludge Drying Area, ft^2 [e]	
Drained beds	
Initial design	22,400
Future design	112,000
Undrained area	
Initial design	62,000
Future design	0

NOTES:

1. Channel width of three feet[d] used for initial installation. For future use, both bar screen channels will be increased to four feet[d] width.
2. One preaeration tank will be used initially to aerate trickling filter effluent prior to secondary sedimentation.
3. For initial design conditions, 200 ft^3/min[f] will be supplied to primary aeration tank and 100 ft^3/min[f] will be supplied to secondary aeration tank.
4. One primary sedimentation tank will be used initially for secondary sedimentation.
5. Future tanks may be constructed with side water depths of 33 ft[d]. Volume of future tanks computed with 33-ft[d] side water depth.

[a] mgd × 0.0438 = m^3/sec

[b] lb × 0.453 = kg

[c] in. × 2.54 = cm

[d] ft × 0.305 = m

[e] ft^2 × 0.093 = m^2

[f] ft^3 × 0.028 = m^3

[g] ft^3/gal × 7.45 = m^3/m^3

[h] gpd/ft^2 × 0.0407 = m^3/day/m^2

[i] lb/1,000 ft^3/day × 0.016 = kg/m^3/day

[j] acres × 0.415 = hectares

[k] lb/acre/day × 1.12 = kg/hectare/day

[l] lb/ft^3/day × 16.2 = kg/m^3/day

[m] ft^3/lb × 0.0624 = m^3/kg

By incorporating nitrification (conversion of ammonia nitrogen to nitrate nitrogen) in the treatment process, free chlorine residuals can be obtained with much lower chlorine dosages. Rock trickling filters, as were used at Livermore, cannot be depended upon to produce a nitrified effluent of the required quality. Used in advance of the activated sludge process, however, trickling filters can reduce the BOD_5 loading sufficiently to allow nitrification to proceed in the activated sludge aeration tank. Utilizing a coupled trickling filtration-activated sludge process at Livermore allowed upgrading of the plant without abandoning any of the existing facilities, which were less than 10 yr old. Using roughing filters also resulted in a more stable overall biological process, with the filters acting as a buffer to protect the more easily upset activated sludge process, an important aspect when high quality effluent must be consistently produced.

TABLE 2. PERFORMANCE OF ORIGINAL PLANT, 1964

Parameter	Value
Influent	
Flow, mgd[a]	2.4
BOD$_5$, mg/l	260
Suspended solids, mg/l	350
Primary Treatment	
Overflow rate, gpd/ft^2 [b]	1,000
Effluent BOD$_5$, mg/l	150
Effluent suspended solids, mg/l	110
Reduction, percent	
BOD$_5$	42
Suspended solids	69
Secondary Treatment	
Organic loading, lb/1,000 ft^3/day[c]	74
Effluent BOD$_5$, mg/l	51
Effluent suspended solids, mg/l	44
Reduction, percent	
BOD$_5$	66
Suspended solids	60
Overall Reduction, Percent	
BOD$_5$	80
Suspended solids	87

[a] mgd x 0.044 = m^3/sec

[b] gpd/ft^2 x 0.041 = m^3/m^2/day

[c] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

At the time that plans were underway to expand and modify the treatment plant, consideration was being given to utilizing plant effluent for irrigation purposes. As noted previously, rapid population growth had depleted local water spplies, and importation of water from other areas had become necessary. Farmlands existed near the treatment plant, and the City was carrying out plans to construct an airport and golf course in the area. Because of these circumstances and because of possible problems with surface discharge of effluent even though highly treated, the City began at that time development of a water reclamation program which has continued and expanded over the years.

Imposition of a "no-discharge" order by the Regional Board required that the plant be designed and constructed as rapidly as possible. A discussion of plant design, with emphasis on utilization of existing facilities, is presented in Section 5.

SECTION 5

DESIGN

When the Regional Board issued new, more stringent discharge requirements in 1965, the original Livermore plant was only 7 yr old. Further, piping and pumping facilities had been constructed to allow exapnsion from 0.11 m^3/sec (2.5 mgd) to 0.22 m^3/sec (5.0 mgd) for the Stage 2 facility and ultimately to 0.44 m^3/sec (10 mgd). These factors encouraged retention of as much of the original facility as possible and allowed inclusion cf a previously planned additional trickling filter in the revised expansion.

Discussion of plant design can be divided into two aspects: process design and physical design. Although these aspects cannot be totally divorced from each other, the differentiation is useful in describing the design of the Livermore plant upgrading. Process design includes the interrelationship among projected influent loadings, required effluent characteristics, anticipated removals in each unit process, and sizing of added unit processes. Physical design is intended to include such factors as general layout, site piping, bypassing procedures, and flexibility for present operation and future expansion. The flow diagram for the expanded plant is shown in Figure 6, and design data are given in Table 3.

PROCESS DESIGN

The principal component of interest at the Livermore Water Reclamation Plant is the biological treatment unit, which uses trickling filtration in series with the activated sludge process followed by secondary clarification. Other important units of the Livermore plant are the primary sedimentation tanks, whose performance determines the loading to the secondary treatment process; the chlorination unit and the chlorine contact tank, which must reduce coliform concentrations to levels meeting disinfection requirements; and solids handling facilities, which at Livermore consist of two mixed digesters followed by sludge lagoons. The design of each of these elements, particularly as they relate to the secondary treatment process, is discussed below.

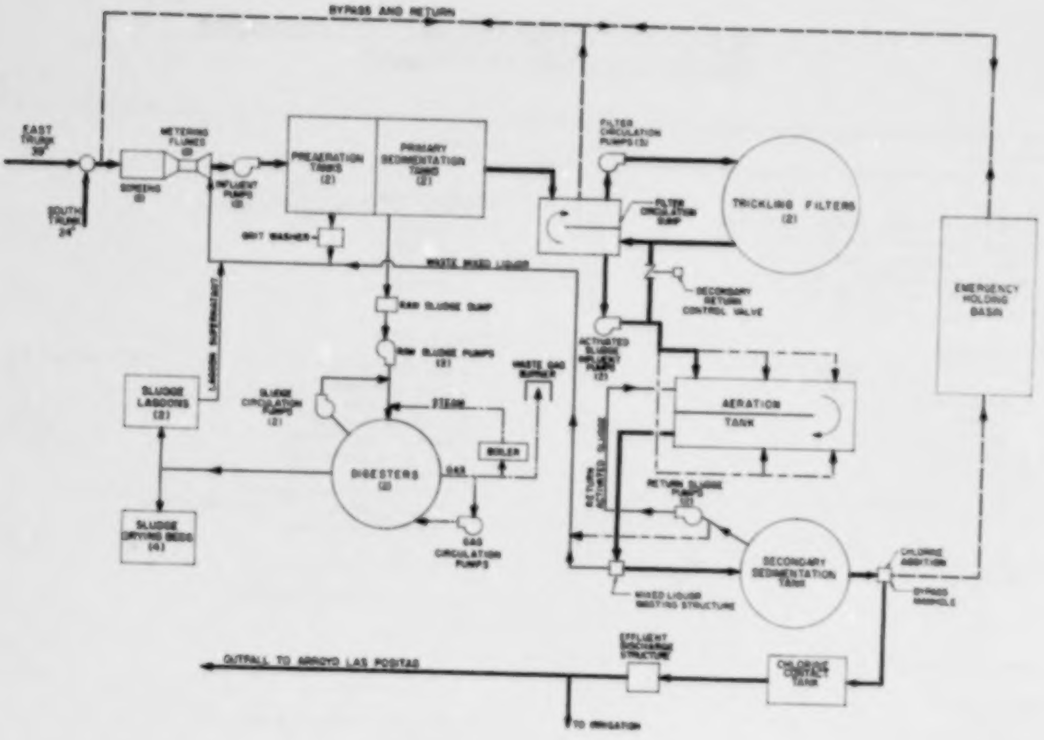

BYPASS AND RETURN
EAST TRUNK 30"
SOUTH TRUNK 24"
SCREENS (2)
METERING FLUMES (2)
INFLUENT PUMPS (3)
PREAERATION TANKS (2)
PRIMARY SEDIMENTATION TANKS (2)
GRIT WASHER
WASTE MIXED LIQUOR
RAW SLUDGE SUMP
RAW SLUDGE PUMPS (2)
LAGOON SUPERNATANT
SLUDGE LAGOONS (2)
SLUDGE CIRCULATION PUMPS (2)
STEAM
WASTE GAS BURNER
BOILER
DIGESTERS (2)
GAS
GAS CIRCULATION PUMPS
SLUDGE DRYING BEDS (4)
FILTER CIRCULATION PUMPS (3)
FILTER CIRCULATION SUMP
TRICKLING FILTERS (2)
SECONDARY RETURN CONTROL VALVE
ACTIVATED SLUDGE INFLUENT PUMPS (2)
AERATION TANK
RETURN ACTIVATED SLUDGE
RETURN SLUDGE PUMPS (2)
MIXED LIQUOR WASTING STRUCTURE
SECONDARY SEDIMENTATION TANK
CHLORINE ADDITION
BYPASS MANHOLE
EMERGENCY HOLDING BASIN
EFFLUENT DISCHARGE STRUCTURE
CHLORINE CONTACT TANK
OUTFALL TO ARROYO LAS POSITAS
TO IRRIGATION

Figure 6. Flow diagram for upgraded Livermore Water Reclamation Plant.

Coupled Trickling Filter-Activated Sludge Process

As noted in Section 4, the controlling factor in process design was the discharge requirement mandating a 5-day median total coliform concentration of 5.0 MPN/100 ml. To prevent excessive chlorine demand, it was essential to oxidize ammonia nitrogen in the wastewater to the nitrate form. Free rather than combined chlorine residuals would then be formed which could provide the required wastewater disinfection at much lower chlorine dosage levels.

In the mid-1960's,nitrification was not commonly used in wastewater treatment plants in the United States and thus nitrification kinetics and proper design criteria were not well documented. The approach taken in design was to use a low organic loading in the biological treatment process to attain nitrification. In developing this approach, it was assumed that trickling filter volume was equally as effective as aeration tank volume in biologically treating wastewater.

TABLE 3. DESIGN DATA FOR UPGRADED LIVERMORE WATER RECLAMATION PLANT

Design Factor	Value
Flow, mgd[a]	
Second stage design, average dry weather	5.0
Second stage design, maximum dry weather	10.0
Second stage design, peak storm rate	18.0
Future design, average dry weather	10.0
Future design, maximum dry weather	19.5
Future design, peak storm rate	36.0
Design Loadings	
Second stage design, population equivalent	62,500
Future design, population equivalent	128,000
Suspended solids, lb/day[b]	
Per capita	0.20
Second stage design	12,500
Future design	25,600
BOD$_5$, lb/day[b]	
Per capita	0.2
Second stage design	12,500
Future design	25,600
Screening Equipment	
Barminutor	
Number	1
Capacity, mgd[a]	14.5
Comminutor	
Number	1
Capacity, mgd[a]	11
Parshall Flumes	
Number	2
Throat width, in.[c]	15
Discharge head, ft[d]	
Two channels, ADWT	0.84
One channel, ADWT	1.3
Raw Sewage Pumps	
Number	2
Capacity each, mgd[a]	12.5
Static lift, ft[d]	11
Preaeration Tanks	
Number	2
Width, ft[d]	19
Length, ft[d]	38
Average water depth, ft[d]	12.7
Detention time, hr	0.6
Air supplied per tank, ft^3/min[e]	150
Hydraulic capacity, mgd[a]	10
Primary Sedimentation Tanks	
Number	2
Width, ft[d]	19
Length, ft[d]	124
Average water depth, ft[d]	9
Effluent weir length per tank, ft[d]	164
Detention time, hr	1.5
Mean forward velocity, fpm[f]	1.4
Overflow rate at ADWT, gpd/ft^2[g]	1,050
Hydraulic capacity, mgd[a]	10
Primary Treatment	
Assumed suspended solids reduction	
Percent	60
lb/day[b]	7,500
Assumed BOD$_5$ reduction	
Percent	35
lb/day[b]	4,400
Trickling Filters	
Number	2
Inside diameter, ft[d]	110
Average filter media depth, ft[d]	4.25
Filter media size, in.[c]	2 to 4
Net area of filter, ft^2[h]	9,500
Volume, 1,000 ft^3/filter[i]	40,000
Circulation ratio to ADWF	1.5 to 3.0
Loading	
Hydraulic loading, gpm/ft^2[j]	0.55
Organic loading, lb BOD$_5$/1,000 ft^3/day[k]	100
Assumed BOD$_5$ reduction	
Percent	50
lb/day[b]	4,000
Trickling Filter Circulation Pumps	
Number	3
Capacity each, mgd[a]	
No.'s 1 and 3	7.6
No. 2	3.8
Average static lift, ft[d]	12.5
Activated Sludge Aeration Tanks	
Number	1
Passes per tank	2
Length per pass, ft[d]	160
Width, ft[d]	30
Average water depth, ft[d]	15
Detention time based on raw sewage flow, hr	5.2
Air supplied, ft^3/lb BOD$_5$ removed[l]	1,200
Volume, ft^3[i]	144,000
Volumetric loading, lb BOD$_5$/1,000 ft^3/day[k]	28
Return sludge, percent	10 to 100
Aeration Blowers	
Number	3
Capacity each, ft^3/min[e]	2,000
Activated Sludge Influent Pumps	
Number	2
Capacity each, mgd[a]	
No. 1	13.5
No. 2	6.2
Average static lift, ft[d]	12.5
Secondary Sedimentation Tanks	
Number	1
Diameter, ft[d]	90
Detention time at ADWF, hr	2.75
Overflow rate at ADWF, gpd/ft^2[g]	787

(continued on next page)

TABLE 3. (continued)

Design Factor	Value
Activated Sludge Treatment	
Assumed suspended solids reduction	
Percent	86
lb/day[b]	4,400
Assumed BOD_5 reduction	
Percent	86
lb/day[b]	3,800
Overall Plant Performance	
Assumed suspended solids reduction, percent	96
Assumed BOD_5 reduction, percent	96
Effluent suspended solids, mg/l	15
Effluent BOD_5, mg/l	15
Chlorine Contact Tank	
Volume, ft^3 [i]	28,000
Detention time at ADWF, hr	1.0
Effluent Cipolletti Weir	
Range, mgd[a]	0 to 15
Size, ft[d]	3
Head at maximum flow, ft[d]	1.75
Emergency Holding Basins	
Storage volume, 1,000 ft^3 [i]	4,140
Detention time at ADWF, hr	150
Digestion Tanks	
Number	2
Inside diameter, ft[d]	35
Side water depth, ft[d]	27.5
Volume, 1,000 ft^3 [i]	53
Unit loading, lb dry solids/ft^3/day	0.22
Assumed volatile matter, percent	75
Volatile matter, 1,000 lb/day[b]	8.8
Assumed volatile matter reduction, percent	60
Volatile matter destroyed, 1,000 lb/day	5.3
Assumed gas production, ft^3/lb volatile matter destroyed[l]	15
Gas produced, 1,000 ft^3/day[i]	79
Assumed solids reduction, percent	40
Digested sludge, 1,000 lb dry solids/day[b]	7.0
Digested Sludge Lagoons	
Number	2
Volume each, 1,000 ft^3 [i]	320
Sludge Drying Beds	
Number	2
Total area, ft^2 [h]	22,400

[a] mgd x 0.0438 = m^3/sec

[b] lb/day x 0.453 = kg/day

[c] in. x 2.54 = cm

[d] ft x 0.305 = m

[e] ft^3/min x 0.028 = m^3/min

[f] fpm x 0.305 = m/min

[g] gpd/ft^2 x 0.0407 = m^3/day/m^2

[h] ft^2 x 0.093 = m^2

[i] ft^3 x 0.028 = m^3

[j] gpm/ft^2 x 0.679 = l/sec/m^2

[k] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

[l] ft^3/lb x 0.0624 = m^3/kg

[m] lb/ft^3 x 16.2 = kg/m^3

An overall loading of 0.56 kg BOD_5/m^3/day (35 lb/1,000 ft^3/day) of aeration tank and/or trickling filter volume was used for design purposes. On this basis, a biological treatment volume of 6,340 m^3 (224,000 ft^3) was required, assuming a plant influent BOD_5 loading of 5,670 kg/day (12,500 lb/day) and a 35 percent BOD_5 removal in the primary sedimentation tanks. Adding the previously planned second trickling filter would provide a total filter volume of about 2,260 m^3 (80,000 ft^3). This meant that an aeration tank volume of 4,080 m^3 (144,000 ft^3) was required to provide the total biological treatment volume of 6,340 m^3 (224,000 ft^3).

Such an approach was simplistic because it assumed an unproven equivalency between filter volume and aeration tank volume and did not recognize that nitrification would occur only in the aeration tank following the trickling filters. It was believed however that the approach was conservative because the series arrangement would provide treatment better than that predicted by considering the volume as a whole.

This approach also eliminated questions concerning feeding an activated sludge unit with trickling filter effluent. The absence of a clarifier between the trickling filters and the activated sludge process presented difficulty in predicting the characteristics of the activated sludge influent (trickling filter effluent). An estimate of 50 percent BOD_5 removal in the filters was employed in the design data shown in Table 3, but there was little documentation to support this value. Further, there was concern that the filter effluent might be less degradable than primary effluent with the same BOD_5 value. Conversely, it was believed that trickling filter effluent BOD_5 would in part reflect biological solids sloughed from the filter media. By considering the trickling filter and activated sludge units as one biological treatment plant, these questions could be effectively ignored.

Since the mid-1960's, the kinetics of nitrification and the criteria for occurence of nitrification in the activated sludge process have become better understood. Operation of the Livermore plant in terms of these considerations, as well as problems caused by lack of adequate knowledge at the time of design, will be discussed more fully in Sections 8 and 10. In particular, it has been found that the air diffusion system capacity was underdesigned, requiring the third blower (origianlly intended to be a standby unit) to be used to attain nitrification.

It is important to note that the flow diagram for the biological treatment unit at Livermore differs from the Activated Bio-Filter, which also combines the trickling filter and activated sludge processes. The Activated Bio-Filter is a proprietary process which is distinguished by return of activated sludge to the trickling filter influent line rather than the head of the aeration tank. The filter thus acts as a reaeration chamber for the activated sludge.

Primary Sedimentation Tanks

The primary clarifiers at Livermore are rectangular and follow preaeration-grit removal tanks in the treatment scheme. Both tanks were constructed with the original plant, and one was used to provide secondary sedimentation following the trickling filter. With the expansion from 0.11 to 0.22 m^3/sec (2.5 to 5.0 mgd), the secondary tank was converted to primary sedimentation as planned, and a new secondary clarifier was constructed. Design removals for the primary clarifiers are 60 percent for suspended solids and 35 percent for BOD_5 at an overflow rate of 42.8 $m^3/day/m^2$ (1,050 gpd/ft^2) and a detention time of 1.5 hr.

Secondary Clarifier

During plant expansion a 27-m (90-ft) diameter secondary clarifier was constructed to provide separation of the activated sludge from the wastewater. The design overflow rate was 32 m^3/day/m^2 (787 gpd/ft^2) at ADWF, which corresponds to 114 m^3/day/m^2 (2,800 gpd/ft^2) at peak wet weather design flows. It was anticipated that such high rates could be avoided by diverting peak flows to the abandoned oxidation ponds, and in fact wet weather flows have been much lower than indicated in Table 3. Nonetheless, it appear that the high loading rates imposed on the secondary clarifier have been the limiting factor in effluent BOD_5 and suspended solids levels achieved by the upgraded Livermore plant. A lower overflow rate would probably be used today. Secondary clarifier performance will be discussed further in later sections.

Disinfection Facilities

Although not part of the original plant, chlorination equipment and a 60-min detention chlorine contact tank were added to the 0.11-m^3/sec (2.5-mgd) facility in 1965 after discharge from the oxidation ponds to Arroyo Las Positas became necessary. During expansion, chlorine feed capacity was increased from 0.11 to 0.22 m^3/sec (2.5 to 5.0 mgd). Although chlorination control by residual measurement through amperometric titration was used at first, this method was abandoned after difficulties were encountered principally with maintenance of the amperometric titration units. Dosage control by pH measurment has been used since. This is possible because the free chlorine residuals which are obtained in a nitrified effluent cause a decrease in pH as chlorine dosage is increased. Although difficulties can occur with this method of chlorination control (other factors may cause the pH to decrease giving a false signal to the chlorination system and halting disinfection), it has been employed with success at Livermore. Further discussion of chlorination control is presented in Section 8.

Solids Handling and Treatment Facilities

The two digesters constructed with the initial facility were retained, and no additional capacity was provided for in the expansion to 0.22 m^3/sec (5.0 mgd). The flow increase, coupled with the addition of the activated sludge process, increased the design unit loadings of the digesters from 1.1 kg dry solids/m^3/day (0.07 lb/ft^3/day) to 3.5 kg/m^3/day (0.22 lb/ft^3/day). To maintain adequate digestion capacity, operation was changed from a primary-secondary mode to primary digestion in both digesters with newly constructed sludge lagoons acting as secondary digesters. The sludge drying beds in the original plant were retained but have rarely been used. Supernatant is periodically withdrawn from the lagoons and returned to the plant headworks. Occasionally the lagoons will have to be dried out and the residual solids removed to landfill. After 8 yr of operation, they have reached their capacity.

Summary

The modifications incorporated in the expansion actually involved relatively minor changes in the flow diagram. The only significant change was the placement of an activated sludge unit between the trickling filters and the secondary clarifier, thereby substantially increasing the biological treatment capacity and (through nitrification) disinfection efficiency of the plant.

PHYSICAL DESIGN

In the same way that the flow diagram was altered relatively little by incorporation of the activated sludge process into the plant, most of the construction involved expansion from 0.11 to 0.22 m^3/sec (2.5 to 5.0 mgd) as previously planned. Influent and primary effluent pumping capacities were increased, the second trickling filter was added, the secondary sedimentation tank was constructed, disinfection capacity was doubled, and sludge lagoons (rather than a planned third digester) were added to increase solids handling capacity. Principal changes from the previously planned expansion involved construction of the aeration tanks, modification of the filter circulation sump to allow its use as both a trickling filter circulation sump and an activated sludge feed sump, construction of a secondary clarifier suitable for settling activated sludge mixed liquor, and abandonment of the oxidation ponds with portions of them being used as emergency holding basins.

Layout of the expanded plant is presented in Figure 7. Comparison with Figure 5 shows little deviation from the previously planned facility. The second trickling filter was constructed as planned to the west of the existing filter. The circular secondary clarifier was placed in the area originally intended for rectangular clarifiers. Space limitations caused by underground piping were a factor in selection of clarifier diameter and, hence, overflow rate.

A two-pass aeration tank was constructed in the area originally intended for the third and fourth trickling filters. Provision was made for future construction of an additional aeration tank to the north end for expansion of the initial tank to the west. Such expansion was anticipated because the two rock filters would not have adequate capacity to receive the ultimate design flow of 0.44 m^3/sec (10.0 mgd). It was expected that biological treatment for future flow increases would be provided by activated sludge only. Recent widespread use of plastic media in trickling filtration, however, indicates

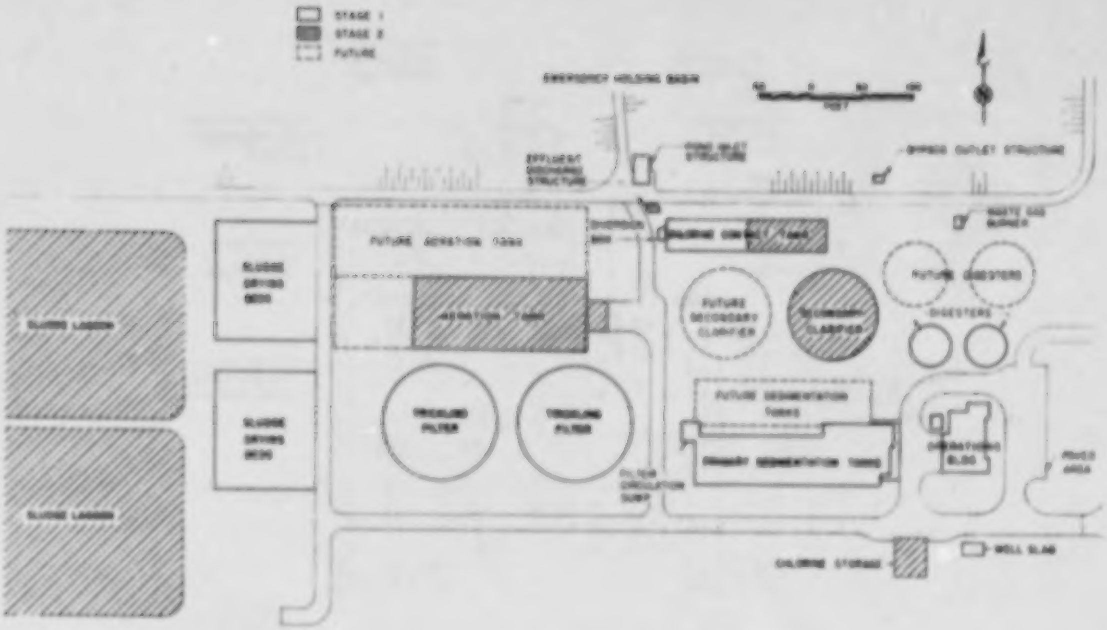

STAGE 1
STAGE 2
FUTURE
EMERGENCY HOLDING BASIN
FEET
EFFLUENT DISCHARGE STRUCTURE
POND INLET STRUCTURE
BYPASS OUTLET STRUCTURE
WASTE GAS BURNER
FUTURE AERATION TANK
DIVERSION BOX
CHLORINE CONTACT TANK
SLUDGE DRYING BEDS
SLUDGE LAGOON
AERATION TANK
FUTURE SECONDARY CLARIFIER
SECONDARY CLARIFIER
FUTURE DIGESTERS
DIGESTERS
TRICKLING FILTER
TRICKLING FILTER
FUTURE SEDIMENTATION TANKS
PRIMARY SEDIMENTATION TANKS
OPERATIONS BLDG
PAVED AREA
SLUDGE DRYING BEDS
SLUDGE LAGOON
FILTER CIRCULATION SUMP
CHLORINE STORAGE
WELL SLAB

Figure 7. Layout of upgraded Livermore plant.

that it may be possible to increase the capacity of the existing rock media filters by replacing the rock with plastic media and increasing the filter depth (such expansion has not yet been carried out, however).

Most of the individual units are of standard design. The following subsections present discussions on three aspects of physical design: (1) the filter circulation sump, (2) the activated sludge unit, including the aeration tank and secondary clarifier, and (3) plant piping, with particular emphasis on bypass procedures.

Filter Circulation Sump

Plan and section views of the original filter circulation sump are shown in Figure 8. Constructed to eventually serve four primary clarifiers and four trickling filters at a design flow of 0.44 m^3/sec (10 mgd), it was desired to retain the unit for use in the coupled trickling filter-activated sludge plant. In the original configuration, primary effluent entered the sump from the channel to the south. Trickling filter effluent to be recirculated entered the

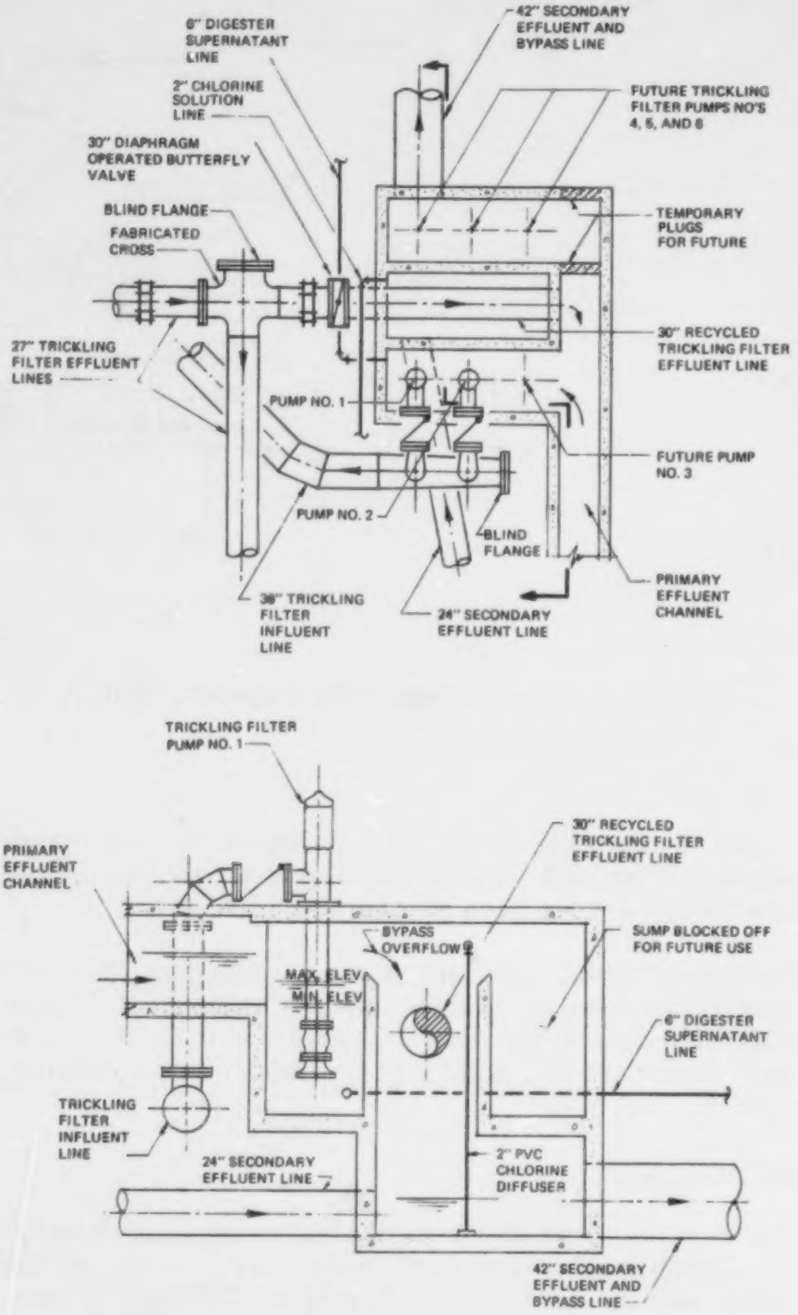

6" DIGESTER SUPERNATANT LINE
2" CHLORINE SOLUTION LINE
30" DIAPHRAGM OPERATED BUTTERFLY VALVE
BLIND FLANGE
FABRICATED CROSS
27" TRICKLING FILTER EFFLUENT LINES
PUMP NO. 1
PUMP NO. 2
36" TRICKLING FILTER INFLUENT LINE
24" SECONDARY EFFLUENT LINE
BLIND FLANGE
42" SECONDARY EFFLUENT AND BYPASS LINE
FUTURE TRICKLING FILTER PUMPS NO'S 4, 5, AND 6
TEMPORARY PLUGS FOR FUTURE
30" RECYCLED TRICKLING FILTER EFFLUENT LINE
FUTURE PUMP NO. 3
PRIMARY EFFLUENT CHANNEL
TRICKLING FILTER PUMP NO. 1
PRIMARY EFFLUENT CHANNEL
30" RECYCLED TRICKLING FILTER EFFLUENT LINE
BYPASS OVERFLOW
MAX. ELEV
MIN. ELEV
SUMP BLOCKED OFF FOR FUTURE USE
6" DIGESTER SUPERNATANT LINE
TRICKLING FILTER INFLUENT LINE
24" SECONDARY EFFLUENT LINE
2" PVC CHLORINE DIFFUSER
42" SECONDARY EFFLUENT AND BYPASS LINE

Figure 8. Filter circulation sump in original plant.

sump from the west through a 0.76-m (30-in.) diaphragm-operated butterfly valve which was controlled by the sump level. Flow to the trickling filter was provided by two fixed-speed pumps of 0.33- and 0.17-m^3/sec (7.6- and 3.8-mgd) capacities. Operation of the pumps was controlled manually from a field station at the sump.

Filter effluent not recirculated was delivered to the secondary clarifier (intended eventually for conversion to primary sedimentation). Effluent from the secondary clarifier was delivered from a blocked-off portion of the primary effluent channel through a temporary 0.61-m (24-in.) line to a concrete box below the filter circulation sump. From there the secondary effluent flowed through a 1.1-m (42-in.) pipe, originally to the oxidation ponds and later to the chlorine contact tank. The concrete box below the filter circulation sump also received primary effluent which overflowed into the box upon shutdown of the trickling filter pumps, thus serving as an automatic bypass unit.

The 0.76-m (30-in.) butterfly valve responded to sump level to maintain a constant flow to the trickling filter by automatically closing as the primary effluent flow increased (causing a sump level increase) and opening as it decreased. It,therefore, allowed use of fixed-speed pumps rather than variable-speed pumps for feeding the trickling filter.

Modifications made to the trickling filter circulation sump during plant expansion are depicted in plan and section in Figure 9. The existing secondary clarifier was converted to a primary clarifier as planned, and the old secondary influent and effluent lines were blocked off. The temporary masonry plug blocking off the north portion of the sump was removed, and two aeration tank feed pumps were installed there. A third trickling filter feed pump with a capacity of 0.33 m^3/sec (7.6 mgd) was also installed.

To permit use of fixed-speed pumps for feeding both the trickling filters and aeration tank, the 0.76-m (30-in.) butterfly valve was relocated on the discharge side of the aeration tank feed pumps as shown. Automatic control is provided by a float switch which causes the valve to close as the level in the sump increases. Normal operation calls for the smaller aeration tank feed pump, with a capacity of 0.27 m^3/sec (6.2 mgd), to run continuously. If the sump level rises above a specified set-point elevation, the larger pump (0.59-m^3/sec or 13.5-mgd capacity) starts and the smaller pump stops. This operation is reversed by a falling sump level.

Shutdown of the trickling filter circulating pumps under this mode of operation results in primary effluent being fed directly to the activated sludge aeration tanks. Shutdown of the aeration tank feed pumps results in overflow to the chlorine contact tank of a mixture of primary effluent and trickling filter effluent.

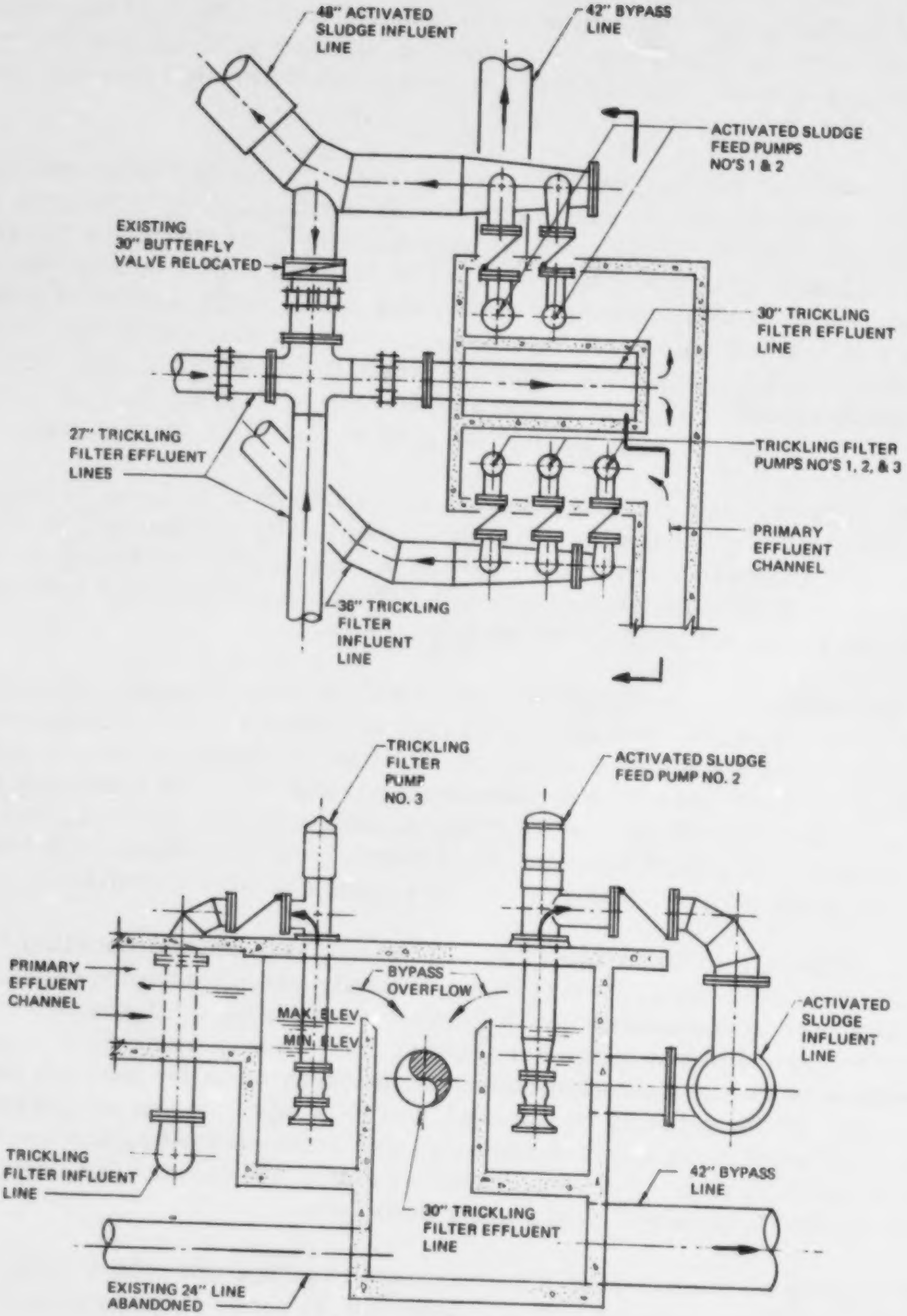

48" ACTIVATED SLUDGE INFLUENT LINE
42" BYPASS LINE
ACTIVATED SLUDGE FEED PUMPS NO'S 1 & 2
EXISTING 30" BUTTERFLY VALVE RELOCATED
30" TRICKLING FILTER EFFLUENT LINE
27" TRICKLING FILTER EFFLUENT LINES
TRICKLING FILTER PUMPS NO'S 1, 2, & 3
PRIMARY EFFLUENT CHANNEL
36" TRICKLING FILTER INFLUENT LINE
TRICKLING FILTER PUMP NO. 3
ACTIVATED SLUDGE FEED PUMP NO. 2
PRIMARY EFFLUENT CHANNEL
BYPASS OVERFLOW
MAX. ELEV
MIN. ELEV
ACTIVATED SLUDGE INFLUENT LINE
TRICKLING FILTER INFLUENT LINE
42" BYPASS LINE
30" TRICKLING FILTER EFFLUENT LINE
EXISTING 24" LINE ABANDONED

Figure 9. Filter circulation sump in upgraded plant.

Schematic drawings of the circulation pumping schemes before and after modification are given in Figure 10. Retention of the existing structure in this manner allowed significant cost savings and avoided abandonment of a facility less than 10 yr old.

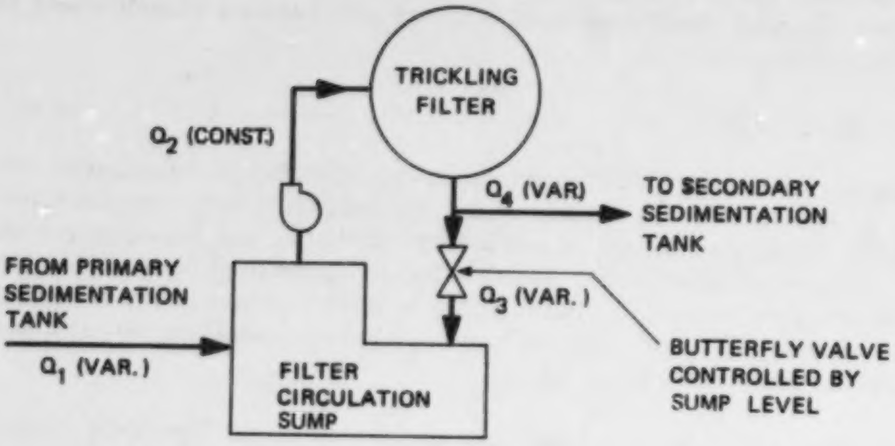
Activated Sludge Unit

The activated sludge unit which receives trickling filter effluent consists of, in addition to the aeration tank influent pumps, a two-pass aeration tank, an air diffusion system, a secondary clarifier, and a sludge return system. The aeration tank was constructed to allow feed of trickling filter effluent and return activated sludge (RAS) at seven points to permit the unit to be operated in plug flow (conventional), sludge reaeration, or step aeration modes as shown in Figure 11.

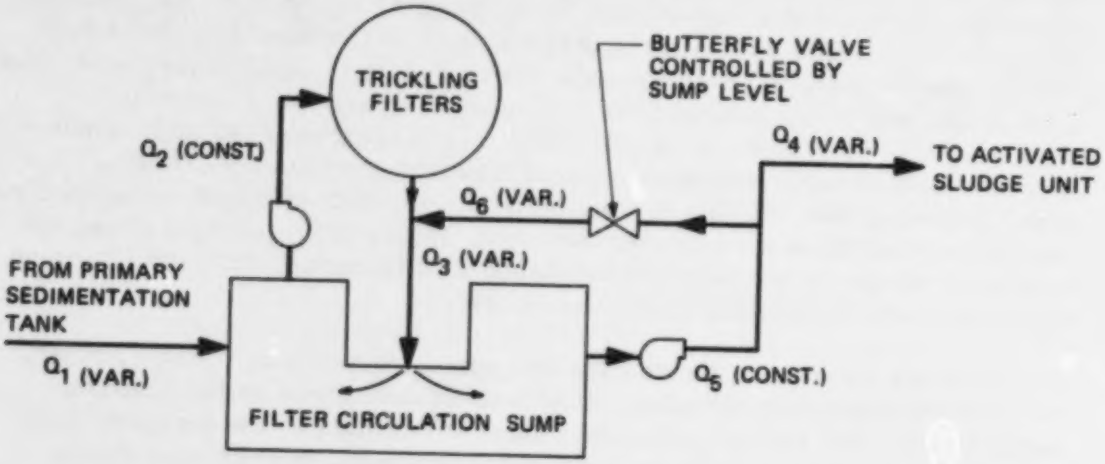
Generally, the plant has been operated in the plug flow mode (although significant back-mixing apparently does occur as discussed in Section 8) with sludge reaeration used for a short period in 1971. Trickling filter effluent and return activated sludge are introduced to the tank at the east end of the south pass. Mixed liquor flows from the south to the north pass through a 4.9-m (16-ft) wide space in the dividing wall at the west end of the tank. Mixed liquor leaves the aeration tank over a 2.7-m (9.0-ft) diameter circular weir at the end of the second pass and flows by gravity to the secondary distribution box. There, an air-lift pump and a constant-head tank combine to waste excess activated sludge mixed liquor to the plant headworks. The rate of mixed liquor wasting is measured by a V-notch weir and is changed by raising or lowering a circular overflow weir (Figure 12). Wasting of excess activated sludge is normally accomplished in this manner although provision has been made for wasting from the RAS line.

From the secondary distribution box,mixed liquor flows to the circular, center-feed secondary clarifier. Clarified effluent flows to the chlorine contact tank, and settled activated sludge is returned to the aeration tank. The RAS pumps are two independent, variable-speed, self-priming pumps which are manually controlled such that the RAS flow is a fixed percentage of the influent wastewater flow. Automatic control could be implemented by using a signal from the influent flowmeter to control pump speed.

Chlorine addition is provided by a diffuser and a flash mixer at the point where clarified effluent enters a 1.4-m (54-in.) reinforced concrete pipe connecting the secondary clarifier with the chlorine contact tank. This line was built as part of the original plant and was used for bypassing from the headworks to the oxidation ponds as discussed below.


TRICKLING FILTER
Q2 (CONST.)
Q4 (VAR)
TO SECONDARY SEDIMENTATION TANK
FROM PRIMARY SEDIMENTATION TANK
Q3 (VAR.)
Q1 (VAR.)
BUTTERFLY VALVE CONTROLLED BY SUMP LEVEL
FILTER CIRCULATION SUMP

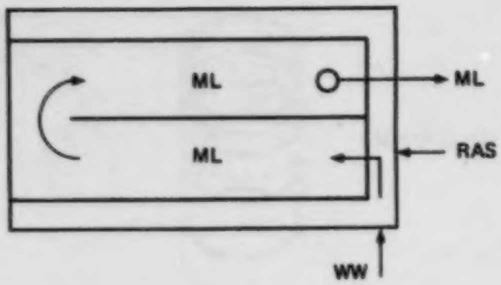
(a) BEFORE MODIFICATION


TRICKLING FILTERS
BUTTERFLY VALVE CONTROLLED BY SUMP LEVEL
Q2 (CONST.)
Q4 (VAR.)
TO ACTIVATED SLUDGE UNIT
Q6 (VAR.)
Q3 (VAR.)
FROM PRIMARY SEDIMENTATION TANK
Q1 (VAR.)
Q5 (CONST.)
FILTER CIRCULATION SUMP

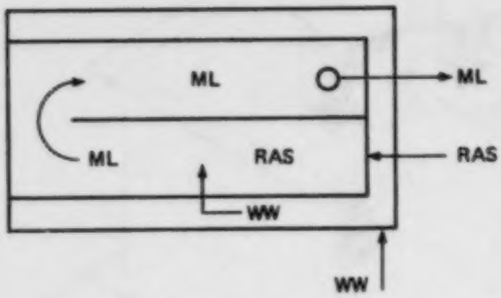
NOTE: SHORT CIRCUITING OF PRIMARY EFFLUENT TO ACTIVATED SLUDGE UNIT WILL NOT OCCUR IF $Q_2 > Q_1$

(b) AFTER MODIFICATION

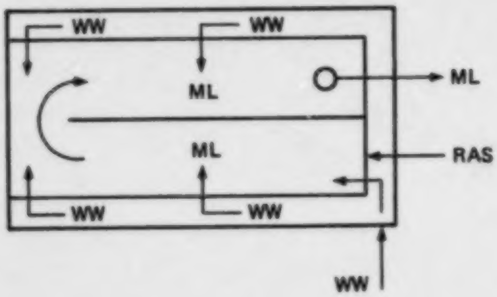
Figure 10. Schematic diagram for filter circulation sump modification.



(a) CONVENTIONAL ACTIVATED SLUDGE



(b) SLUDGE REAERATION



(c) STEP AERATION

NOTE: ML = MIXED LIQUOR
RAS = RETURN ACTIVATED SLUDGE
WW = INFLUENT WASTEWATER

Figure 11. Possible modifications for activated sludge feed.

Plant Piping

A schematic piping diagram for the original plant, including anticipated expansions to 0.22 and 0.44 m^3/sec (5.0 and 10.0 mgd) is shown in Figure 13. When the decision was made to incorporate the activated sludge process into the upgraded treatment scheme, it was desired to retain as much of the existing piping as possible to reduce costs. In addition, the stringent discharge requirements dictated that in the event of a system upset or unit shutdown, some means must be available to prevent discharge of inadequately treated effluent. To accomplish this, emergency holding basins were constructed from a portion of the original oxidation ponds and flows were diverted from the plant when necessary.

The piping diagram for the upgraded plant is presented in Figure 14. (In Figures 13 and 14, only the main flow and bypass lines are shown; solids transfer piping and various other lines such as chlorine lines and drains are not indicated.) Principal modifications are the addition of lines from the filter circulation sump to the aeration tank, from the aeration tank to the secondary clarifier, from the filter circulation sump to the second trickling filter and back, and from the existing bypass line to the emergency holding ponds. The existing 1.4-m (54-in.) bypass line was used to carry secondary effluent from the secondary clarifier to the chlorine contact tank and the newly constructed outfall to Arroyo Las Positas.

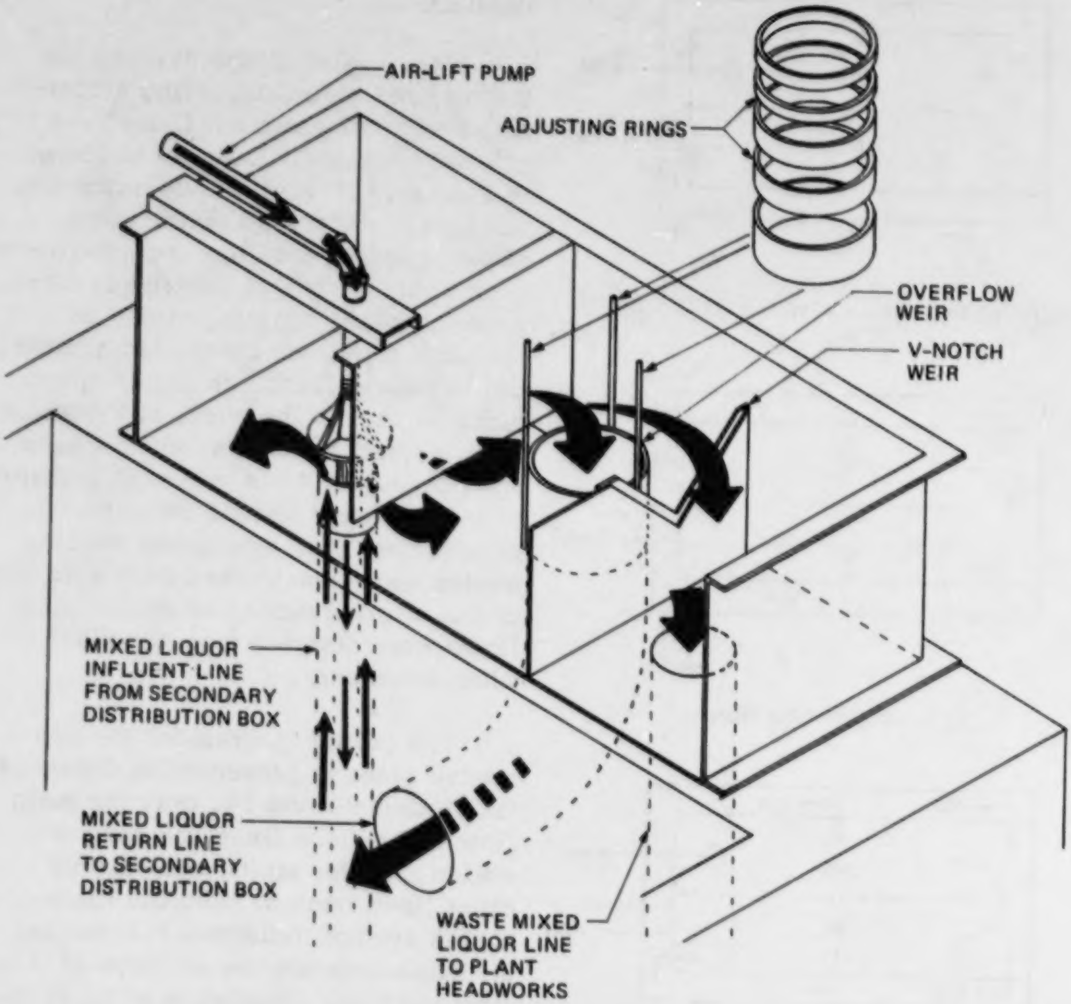

AIR-LIFT PUMP
ADJUSTING RINGS
OVERFLOW WEIR
V-NOTCH WEIR
MIXED LIQUOR INFLUENT LINE FROM SECONDARY DISTRIBUTION BOX
MIXED LIQUOR RETURN LINE TO SECONDARY DISTRIBUTION BOX
WASTE MIXED LIQUOR LINE TO PLANT HEADWORKS

Figure 12. Mixed liquor wasting tank.

Bypassing procedures in the upgraded plant differ from those used in the original configuration. In the trickling filter-oxidation pond mode, raw sewage or primary effluent could be directed to the oxidation ponds for a substantial period without overloading them. Further, because the ponds would themselves provide adequate treatment, bypassed sewage did not need to be returned to the plant.

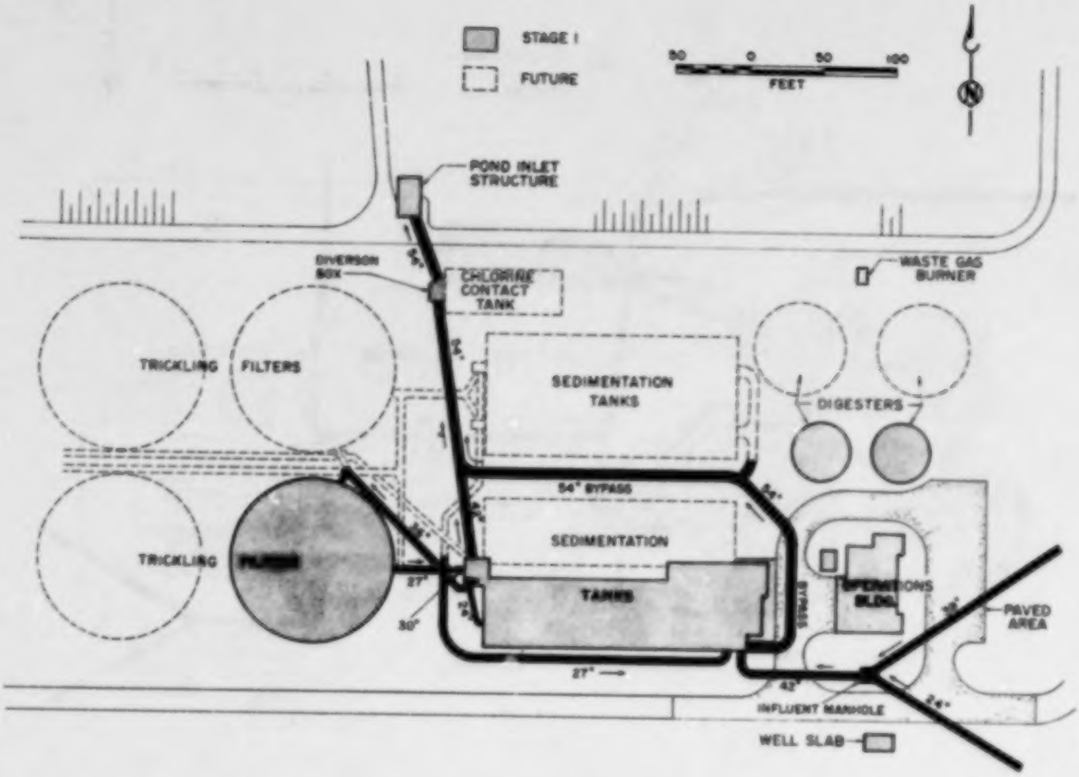

STAGE I
FUTURE
50
0
50
100
FEET
N
POND INLET STRUCTURE
DIVERSON BOX
CHLORINE CONTACT TANK
WASTE GAS BURNER
TRICKLING FILTERS
SEDIMENTATION TANKS
DIGESTERS
54" BYPASS
SEDIMENTATION
TRICKLING FILTER
TANKS
27"
30"
27" →
BYPASS
OPERATIONS BLDG.
PAVED AREA
42"
INFLUENT MANHOLE
WELL SLAB

Figure 13. Piping diagram for original plant.

In the upgraded mode, bypassing of the treatment units to Arroyo Las Positas cannot be allowed if discharge requirements are to be met. Therefore, when bypassing is necessary, flow is directed to the emergency holding basins; when plant operation is restored, wastewater is returned by gravity to the headworks and passes through all the process units. From the headworks (raw sewage) or secondary clarifier (secondary effluent), wastewater flows to the holding basins through the existing 1.4-m (54-in.) bypass line and a newly constructed 1.1-m (42-in.) line. From the filter circulation sump (primary effluent or trickling filter effluent), bypass flow passes through the old 1.1-m (42-in.) and 1.4-m (54-in.) lines connecting the sump with the original pond inlet structure. An important operational aspect of the latter case is that the poorly treated wastewater must pass through the chlorine contact tank. It has been found that this contaminates the tank and creates subsequent difficulties in meeting disinfection requirements unless the tank is cleaned prior to being restored to service.

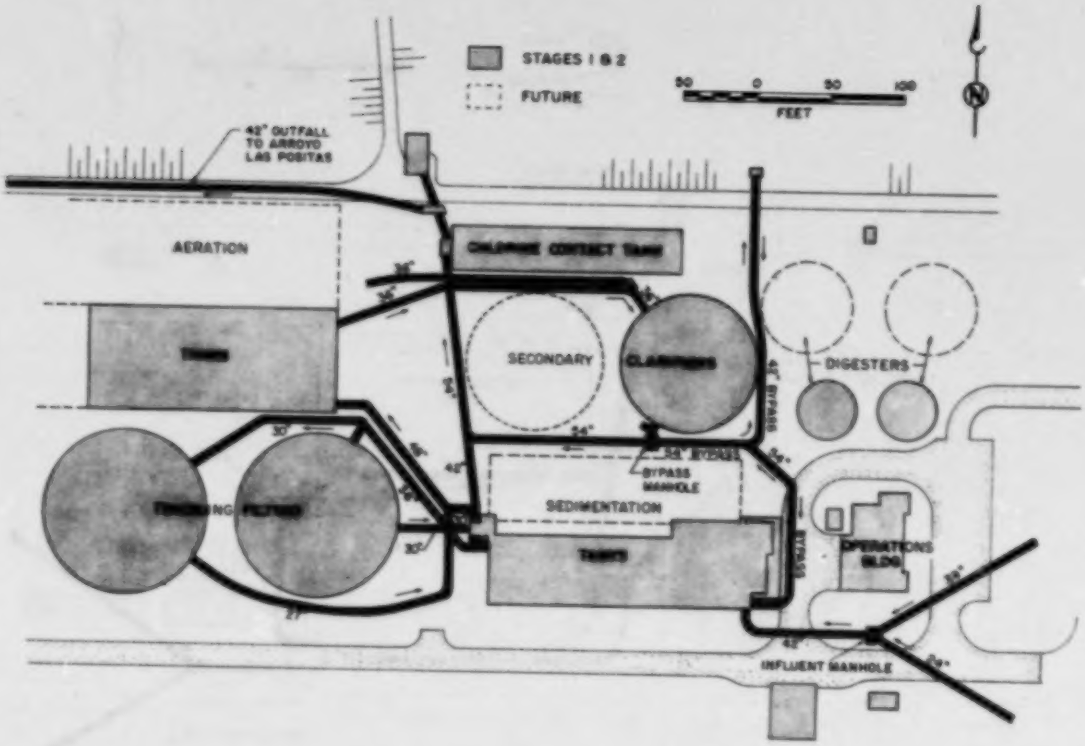

STAGES 1 & 2
FUTURE
50 0 50 100
FEET
N
42" OUTFALL TO ARROYO LAS POSITAS
AERATION
TANKS
CHLORINE CONTACT TANK
36"
36"
SECONDARY
CLARIFIERS
DIGESTERS
42" BYPASS
24"
BYPASS MANHOLE
SEDIMENTATION
TANKS
TRICKLING FILTERS
30"
42"
30"
27"
BYPASS
OPERATIONS BLDG
42"
30"
INFLUENT MANHOLE

Figure 14. Piping diagram for upgraded plant.

The emergency holding ponds at Livermore are required in part because the activated sludge process consists of one unit rather than two parallel units. While shutdown of one of the two primary sedimentation tanks or trickling filters can be accomplished without a complete plant shutdown, this cannot be done with the aeration tank or the secondary clarifier; maintenance on or repair of either unit requires bypassing of plant flow to the holding basins. Because it is less expensive to construct one unit at a given capacity than two parallel units, each at half capacity, existing ponds or sludge lagoons should always be considered for emergency holding basins in upgrading an existing plant.

The flow received by the Livermore plant is presently nearing its 0.22-m^3/sec (5.0-mgd) capacity. One of the alternatives being considered for expansion is to incorporate flow equalization into plant operation through the use of existing bypass lines and holding ponds, one of which has been lined. It is anticipated that a flow of perhaps 0.26 m^3/sec (6.0 mgd) could be accommodated in this manner.

SECTION 6

CONSTRUCTION

Construction as well as design of a sewage treatment facility is much simpler and more straightforward when dealing with a completely new treatment plant than when upgrading an existing plant. Construction schedules in the latter case are tied to and in some instances controlled by treatment plant operating procedures. This was less of a problem at Livermore than it often is, but there was the additional operating constraint of the Regional Board order which prohibited discharge of inadequately treated effluent to surface waters. Thus, it was imperative that construction be completed as quickly as possible and that there be close cooperation between the Contractor and plant personnel.

PRECONSTRUCTION ACTIVITIES

At the completion of the design phase, construction drawings and specifications were prepared. These contract documents were approved by the City of Livermore and by the San Francisco office of the U. S. Department of Health, Education, and Welfare (HEW). The City had obtained a federal grant, No. WPC-CAL-258, to partially finance construction of the upgraded facilities, and HEW's consent was necessary prior to advertising for bids.

The City Council authorized the construction of the trickling filter-activated sludge facility, and the contract documents, labeled "Treatment Plant Enlargement - 1965", were advertised for bid on January 24, 1966. Copies were made available to prospective bidders and were distributed to builder's exchanges in the San Francisco Bay Area.

The requested construction bid was to include the furnishing of all labor, materials, and equipment for construction of the second trickling filter, aeration tank, secondary sedimentation tank, distribution structure, bypass system, effluent discharge facilities, chlorination additions, outfall sewer, and sludge lagoons; modifications and alterations to existing headworks, primary sedimentation tanks, filter circulation sump, and pond distribution structure; paving, grading, painting, piping, and fencing; and all other related work necessary to provide a complete and operable treatment plant.

Normal prebidding activity was experienced as reflected by the number of plan holders, a typical situation in California for a project of this size. Two addenda to the documents were issued during this period. One was related to revised prevailing wages as determined by the U. S. Department of Labor; the other was related to technical clarifications in the plans and specifications.

The contract documents requested bids on two alternatives: Bid Item I included the improvements as shown; Bid Item II included the improvements as shown with the omission of the airport effluent disposal irrigation system. Included in each item was the sum of $20,000 for contingencies encountered during construction. Bids were opened on February 24, 1966, and the results are summarized in Table 4.

TABLE 4. BID TABULATION

Name of Bidder	Bid Item I	Bid Item II
Pacific Mechanical Corp.	$1,057,800	$ 985,400
Monterey Mechanical Co.	1,099,000	1,032,000
Fred J. Early Jr. Co.	1,126,800	1,049,800
C. Norman Peterson Co.	1,130,500	1,059,399
S. & Q. Construction	1,134,399	1,085,000

The low bid submitted by Pacific Mechanical Corporation was found to be in order, and the contract was awarded to that company. Approval from HEW to sign the contract with the Contractor was obtained by the City on February 28, 1966, and the notice to proceed was issued by the City Engineer on March 16, 1966. The allowable construction period was 300 days, and the designated completion date was January 10, 1967.

CONSTRUCTION ACTIVITIES

Policies on construction management were finalized soon after the notice to proceed was issued to the Contractor. The City's consultant, Brown and Caldwell, was asked to assist in the review of shop drawings submitted by the Contractor and to provide periodic field support services as required by the City. The City was to provide all required inspection services and be responsible for all change orders, progress payments, interpretation of contract documents, and contact with HEW officials. Whenever required, Brown and Caldwell engineers would also be available for clarification of contractual matters, especially the technical provisions.

The contract documents made provisions for listing the major pieces of equipment in the proposal, both by name and installed cost. Pacific Mechanical Corporation submitted the equipment listed in Table 5.

The low bid had been based on the lowest installed price for each item of major equipment. The City made its equipment choices known to the Contractor on March 14, 1966. The selected items were: 1) Armco,

TABLE 5. MAJOR EQUIPMENT ITEMS BREAKDOWN

Item	Article in Specifications	Manufacturer	Installed Price, dollars
1	Sluice gates	Armco	22,500
		Rodney Hunt	25,000
2	Trickling filter circulating pump	Fairbanks-Morse	3,750
		Johnston Pump	3,500
		Worthington	3,500
3	Aeration pumps	Fairbanks-Morse	12,000
		Johnston Pump	12,000
		Worthington	13,000
4	Air diffusion equipment	Chicago Pump	35,000
		Walker Process	40,000
5	Air blowers	Roots-Connersville	40,000
		Sutorbilt	45,000
6	Secondary sedimentation tank equipment		
	Alternate A	Chain Belt	50,000
	Alternate B	Walker Process	40,000
	Alternate C	Walker Process	30,000
7	Trickling filter distributor	Dorr-Oliver	20,000
		Walker Process	19,000
8	Chlorination equipment	Wallace & Tiernan	19,000
		Fischer & Porter	20,000

2) Johnston Pump, 3) Johnston Pump, 4) Chicago Pump, 5) Roots-Connersville, 6) Alternate B: Walker Process, 7) Walker Process, and 8) Wallace & Tiernan. Alternate B of item 6 was not the lowest installed price, but it was chosen because it provided a sight well which was considered of prime importance for proper plant operation. The choice of Alternate B added an extra $10,000 to the low bid, and the revised construction cost became $1,067,800. Approval from HEW for this revised cost was obtained on March 18, 1966.

The Contractor's low bid was a lump sum figure. For cash flow analysis the Contractor submitted, as required in the contract documents, a breakdown of his total bid together with his estimated progress schedule. The bid breakdown, summarized in Table 6, was used to determine the monthly progress payment estimates.

Before describing the sequence of events during construction, it is worthwhile to briefly review the items constructed, with emphasis on the mechanical equipment and construction details.

TABLE 6. CONTRACT PRICE BREAKDOWN

Description	Amount, dollars	
Job Mobilization		
(Purchasing, Engineering, Move On)		$ 12,000.00
Earthwork		
Site grading	5,000.00	
Structure excavation	34,100.00	
Sludge lagoons	27,500.00	
Outfall excavation	17,600.00	
Piping and miscellaneous excavations	27,500.00	
Oxidation pond	36,500.00	
Backfill fine grading	9,100.00	
Paving	6,900.00	
Total		164,200.00
Forming		
Secondary sedimentation tank	16,000.00	
Aeration tank	36,100.00	
Trickling filter	26,300.00	
Chlorine contact tank	14,100.00	
Miscellaneous structures	16,500.00	
Total		109,000.00
Reinforcing Steel		
Delivered to job site	21,500.00	
In place	26,500.00	
Total		48,000.00
Concrete		
Secondary sedimentation tank	18,200.00	
Aeration tank	42,000.00	
Trickling filter	16,500.00	
Chlorine contact tank	7,600.00	
Miscellaneous structure	12,700.00	
Total		97,000.00
Miscellaneous Metal Work		
Embedded work	13,000.00	
Other items	36,000.00	
Total		49,000.00
Equipment		
Delivery on site	200,000.00	
Installation	11,800.00	
Total		211,800.00
Piping		
Material at job site	152,000.00	
Outfall sewer	14,700.00	
Bypass and interconnecting sewer	3,400.00	
Miscellaneous yard piping	23,500.00	
Total		193,600.00
Trickling Filter Block and Rock		22,000.00
Other Subcontracts		
Electrical	52,000.00	
Sprinklers	63,000.00	
Boring	14,700.00	
Fencing	2,500.00	
Painting	7,300.00	
Total		139,500.00
Move Off and Clean Up		1,700.00
Contingency		20,000.00
TOTAL		$1,067,800.00

Trickling Filter

The added second filter is of reinforced concrete construction and equivalent to filter No. 1, 34 m (110 ft) in diameter with a 1.2-m (4.0-ft) deep rock media layer. The maximum rock size is 10 cm (4 in.), and the media rests on Process Engineers' "Air In" underdrain concrete blocks. The filter floor slab slopes towards a central collecting channel which also slopes towards an effluent collecting pipe. The filter walls have rectangular openings at floor slab elevation for air circulation purposes. Four reaction type distribution arms, "Rotoseal" models manufactured by Walker Process Company, distribute the flow through 5-cm (2-in.) orifices to the filter media. Spread deflectors at each orifice provide for even distribution.

Filter Circulation Pump

The No. 3 pumping unit added to the filter recirculation system is a mixed flow, Model 16 LS pump provided by Johnston Pump Company with a capacity of 0.33 m^3/sec (7.6 mgd) at 5.3 m (17.5 ft) total dynamic head (TDH). The overall pumping capacity is as follows: pumps Nos. 1 and 3, 0.33 m^3/sec (7.6 mgd) each; pump No. 2, 0.17 m^3/sec (3.8 mgd).

Aeration Tank

The reinforced concrete aeration tank is 49 m (160 ft) long and includes two passes, each 9.1 m (30 ft) wide. Flow distribution channels along the outside edge of each pass include openings for removable slide gates. This arrangement provides for three modes of activated sludge operation as discussed in Section 5: conventional, step aeration, and sludge reaeration. Coarse-bubble air diffusers, located along each side of the central wall, are Chicago Pump's Type B1 Swingfuser Aerators. The air manifold feeds eight diffuser assemblies in each pass, and each assembly contains 20 synthetic resin "deflectofusers" with four 0.79-cm (5/16-in.) diameter jet openings. Each diffuser includes a 1.35-cm (17/32-in.) control orifice for equal air distribution. The diffuser manifolds are located 0.61 m (2.0 ft) above the floor slab and 0.81 m (2.67 ft) from the central wall. The diffusers are placed onto the central-wall side of the manifolds in a staggered arrangement (Figure 15). The diffuser assemblies can be lifted out of the tank by using a 1-ton portable manual winch and hoist. The distribution channels are also equipped with air diffusers.

Aeration air supply units are installed adjacent to the tanks on a concrete slab at ground elevation and include three rotary lobe, positive displacement, Model 1021 RAS Roots-Connersville blowers, complete with inlet silencers with filters and outlet silencers. Each blower has a capacity of 56.6 m^3/min

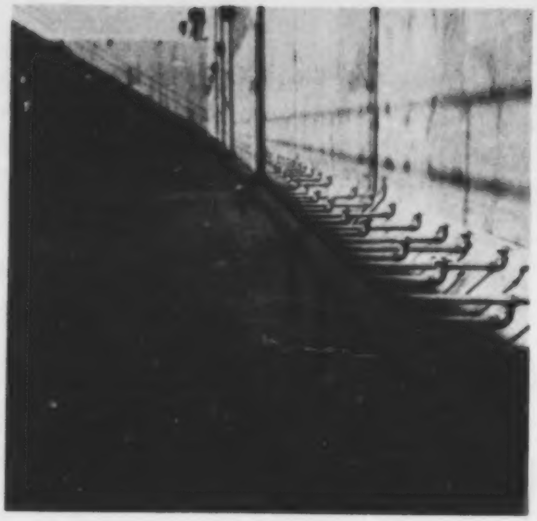


Figure 15. Aeration diffusers.

(2,000 ft^3/min) at 48 kN/m^2 (7 psi). Discharge from the blowers is manifolded into a header pipe which supplies the air header atop the central wall of the aeration tank.

Aeration Tank Feed Pumps

Aeration pumps, to lift trickling filter effluent from the filter circulation sump to the aeration tank, are located in the filter distribution structure opposite the trickling filter circulation pumps and are manufactured by Johnston Pump Company. Pump No. 1 is a propeller type, Model 20 PO, and has a capacity of 0.59 m^3/sec (13.5 mgd) at 3.2 m (10.5 ft) TDH. Pump No. 2 is a mixed flow, Model 16 LS pump and has a capacity of 0.27 m^3/min (6.2 mgd) at 3.2 m (10.5 ft) TDH.

Secondary Sedimentation Tank

The reinforced concrete secondary sedimentation tank is 27 m (90 ft) in diameter with a 3.7-m (12-ft) side water depth (SWD). It is of the center-feed type and is equipped with a Type SWP multidraw collector by Walker Process Equipment Company, complete with "Clariflow" inlet, sight well, and truss arms. Suction pipes in the revolving truss arms are connected to a water-level sight well and are provided with telescopic valves for flexibility in sludge draw-off. The revolving arms are also equipped with adjustable brass squeegees for sweeping the sludge on the floor slab to the withdrawal pipes. Secondary effluent flows over a weir plate to a collection channel inside the tank perimeter. A skimmer assembly, mounted on the rotating truss, sweeps surface scum to a scum box.

Return Activated Sludge Pumps

Return activated sludge units are pad-mounted next to the secondary sedimentation tank and include two variable-speed, motor-driven pumps and a magnetic flowmeter. The pumps are Gorman-Rupp Model T8A-3, each with a capacity of 0.1 m^3/sec (2.3 mgd) at 9.1 m (30 ft) TDH. Electric Machinery's "Ampli-Speed" magnetic drives, Model MDM-13, provide speed variation. Pump suction is taken from the bottom center of the secondary sedimentation tank. The pump discharge lines are manifolded to a common header (containing the flowmeter) which ends at the inlet to pass No. 1 in the aeration tank.

Chlorination System

New facilities were added and the existing chlorine contact tank modified to increase plant effluent disinfection capability. New facilities include a chlorine storage area, an evaporator, and chlorine residual analyzers. The chlorine contact tank capacity was doubled.

An open steel structure, with a Robertson Q-Type deck (supported by structural steel members) serving as a roof, and a concrete floor slab provide storage for eighteen 907-kg (2,000-lb) chlorine cylinders. For cylinder handling and weighing,an 1,814-kg (4,000-lb) electric hoist mounted on an overhead monorail and two load cells were furnished.

A Wallace & Tiernan Series A-785 evaporator with a capacity of 1,8 14 kg/day (4,000 lb/day) was installed in the existing chlorinator room in the operations building. The existing Wallace & Tiernan Series A-711 chlorinators were modified to accept a signal from the chlorine residual analyzer controller in addition to the flow signal.

Wallace & Tiernan chlorine analyzers, complete with Moyno Model IL6CDQ sampling pumps, were installed at the effluent discharge structure, downstream of the chlorine contact tank,and downstream from the bypass manhole in the secondary effluent pipeline just outside the secondary sedimentation tank. The postchlorination application point is at the bypass manhole, which is equipped with a turbine type mixer made by Cleveland Mixer Company.

The existing 4.6-m (15-ft) deep, 3.2-m (10.5-ft) wide, two-pass,concrete chlorine contact tank was lengthened by 15.6 m (51 ft). Hinde Engineering's Air-Aqua Oxidation System 2542 air tubing was installed in a serpentine pattern on the floor slab of both the old and new tank sections.

Effluent Disposal Lines

A 1.1-m (42-in.) concrete sewer line 701 m (2,300 ft) long was installed from the effluent discharge structure to the Arroyo Las Positas Creek bed for effluent disposal. Diversion of plant effluent is provided for at two manholes, either for airport spray irrigation or for golf course irrigation and storage. The airport system includes a pumping station with two four-stage Model 12 AC turbine pumps manufactured by Johnston Pump Company, each with a capacity of 1.9 m^3/min (500 gpm) at 67 m (220 ft) TDH; sprinkler controllers; pop-up type sprinkler heads; and approximately 20 ha (50 ac) of land. The land is located mostly south of the Livermore Municipal Airport runway, which is about 0.8 km (0.5 mi) west of the treatment plant.

A 0.36-m (14-in) diameter, 1,158-m (3,800-ft) long pipeline was constructed for effluent diversion to the Las Positas Golf Course adjacent to and northwest of the airport. A motor-operated butterfly valve in the upstream end of the pipeline is responsive to water surface elevation in four interconnecting storage lakes which supply the golf course irrigation system.

Miscellaneous Facilities

Miscellaneous items constructed under this contract included pipelines interconnecting different process units, asphaltic concrete pavement, storm drains, chain link fence, and landscaping, including a sprinkler system. Miscellaneous equipment added during plant enlargement included a Worthington Model 25-C-5 comminutor located in the inlet works channel just upstream of the raw sewage pumps; a Chicago Pump Tru-Test effluent sampler complete with a Gorman-Rupp Model 81-1/2-B2-B sampler pump, both located on the effluent discharge structure top slab; two No. 3 spray water pumps by Johnston Pump Company, Model 7BC, each with a capacity of 1.1 m^3/min (280 gpm) at 15 m (50 ft) TDH, also located on the effluent discharge structure slab and used to control foaming in the aeration tank; a 0.9-m (36-in.) sluice gate with a hydraulic operator at the pond inlet structure for plant effluent bypass; and other small items. Electrical and instrumentation work and painting services were also provided by the Contractor under the terms of the contract.

CONSTRUCTION SEQUENCE

Construction management was directed by the Livermore City Engineer. At the job site he was represented by a City inspector who devoted his full time to construction supervision. In the absence of the inspector (illness, etc.), the treatment plant superintendent divided his duties between construction inspection and operation. Change orders and requests for progress payments originated with the inspector and were submitted to the City Engineer for approval. Change orders, in turn, were submitted by the City to HEW for approval. Shop drawings, reviewed by Brown and Caldwell, were handled and distributed by the City. Brown and Caldwell engineers were also available for direct consultation with the City inspector and made periodic visits to the job site to assist in the inspection procedures. On request, Brown and Caldwell assisted the City Engineer in his dealings with the Contractor.

Record keeping, in addition to correspondence between the Contractor and the City, included the preparation of a daily report by the inspector from March 16 to December 15, 1966; construction progress photographs taken once a month from March 1966 through November 1966; soil and concrete test reports; field trip reports prepared by the consulting engineering firm; and general correspondence between the City and its consultant.

Weather had little effect on construction progress as the climate of the Livermore area is semiarid, with hot, dry summers and cool, moist winters. Average summer temperatures are above 21 C (70 F) and winter temperatures are above 7 C (45 F). Rainfall during the period of intensive construction, March to December 1966, was below average. Total yearly rainfall for 1966 was 25.98 cm (10.23 in.), whereas the long-term average based on the 1931-1960 period is 36.58 cm (14.40 in.) per year. Construction activities were not seriously curtailed during the rainfall period; there were no job shutdowns because of inclement weather. There was, however, substantial reduction of the labor force occasionally because of wet ground. Monthly precipitation and evaporation values for January 1966 through April 1967 are shown in Table 7.

TABLE 7. RAINFALL AND EVAPORATION DURING CONSTRUCTION PERIOD

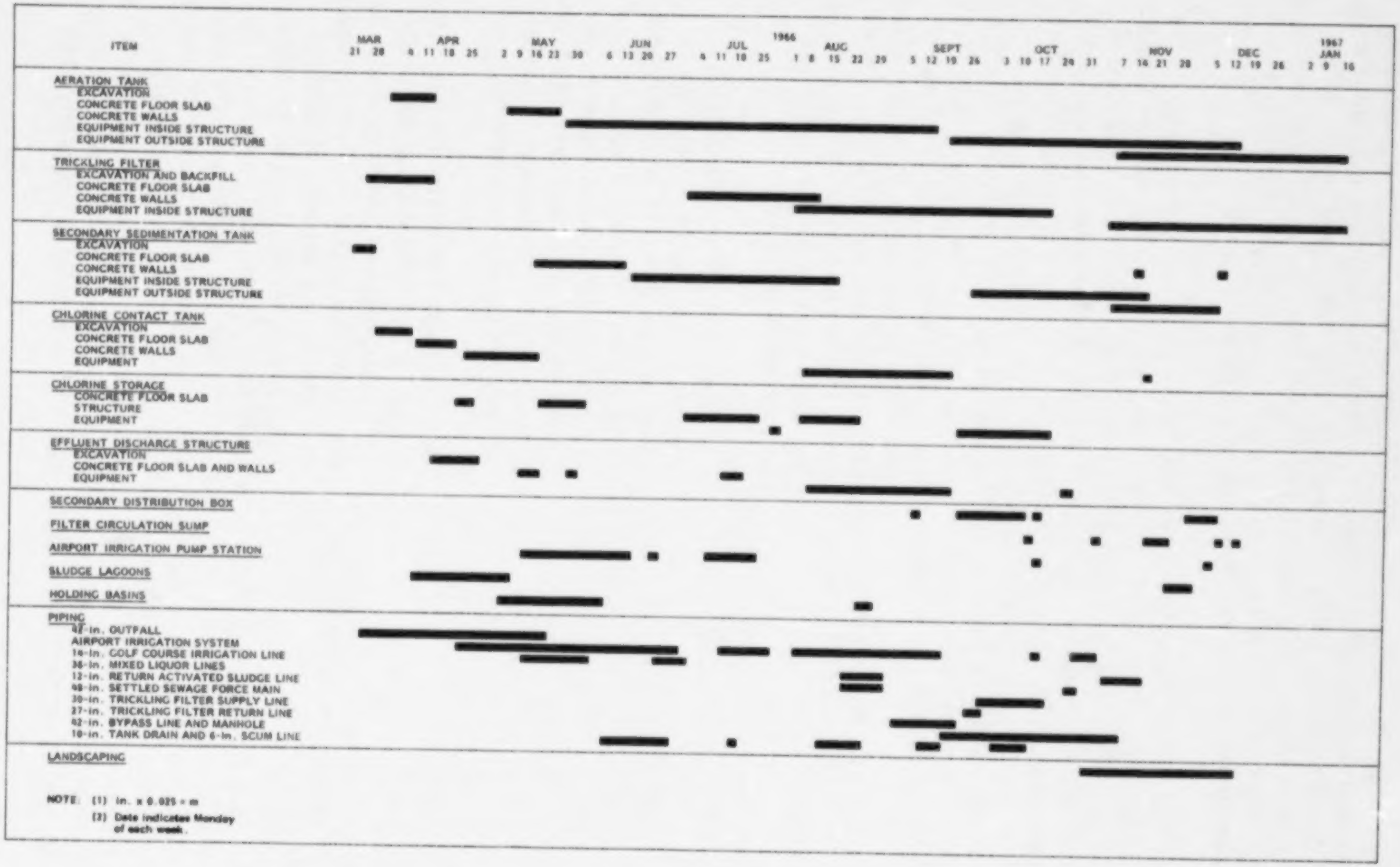
Month	Rainfall, in.[a,b]	Evaporation, in.[a,b]
January 1966	1.63	1.61
February	0.97	2.26
March	0.20	3.82
April	0.36	7.76
May	0.15	9.55
June	0.10	10.31
July	0.18	10.88
August	0	11.93
September	0.11	8.0
October	0	6.5
November	3.75	3.0
December	2.78	1.18
January 1967	7.44	2.25
February	0.28	2.18
March	5.12	3.22
April	4.31	3.19

[a] Recorded at Livermore Water Reclamation Plant.

[b] in. x 2.54 = cm

The Contractor's activities started on March 16, 1966, when his construction quarters were moved to the job site. Ground alignment, both vertical and horizontal, was established by the City the next day. Four days later, the Contractor's earthmoving equipment was at work.

Although the entire construction period is not included in the inspector's daily reports (as they cover only the period from March 16 to December 15, 1966), his reports do include information on the bulk of construction activities. This diary along with other field information has been used to establish the Contractor's activities as summarized in Figure 16. The approximate period covered by each activity, excluding the minor breaks or pauses which exist normally in any construction project, is shown graphically. Figure 16 cannot be used, however, to explain why some activities took longer than expected to complete. To help visualize the construction progress and its many ramifications, each activity is briefly discussed. The



NOTE: (1) in. x 0.025 = m
(2) Date indicates Monday of each week.

Figure 16. Construction activity.

relationship of the different units to each other and the location of the piping systems serving those units were presented previously in Figures 7 and 14 (Section 5), respectively.

Aeration Tank

The proximity of the trickling filter and sludge lagoons to the aeration tank helped coordination of earthwork activities at these three sites. Material excavated from the aeration tank site was used for engineered fill at the trickling filter structure and for dikes at the sludge lagoons. Embankment compaction values ranged from 97 to 99 percent relative density.

The quantity of concrete poured in this structure totaled nearly 1,150 m^3 (1,500 yd^3); the largest single pour was the floor slab at 516 m^3 (675 yd^3). The maximum size of the aggregate for this large pour was 3.8 cm (1.5 in.), and a bucket-conveyor belt combination was used to pour the entire floor area. For the remainder of the tank, the aggregate size was reduced to 1.9 cm (0.75 in.), and the concrete was pumped using a Thompson Ready-Mix pump. During concrete form construction and rebar placing, the walls were divided into 12-m (40-ft) sections for each pour and alternate sections were cast simultaneously, starting at the south wall and ending at the west wall. Probably because of problems with the concrete pump and mix (three truckloads were rejected), large rock-pockets developed at the east end of the center wall. All loose material was removed in the affected area, and an epoxy compound was used to bond the new concrete to the existing surfaces.

Equipment delivery problems adversely affected the installation of the air blowers. Blowers were air-freighted and installed early in 1967. Because blower motor delivery could not be made until March 1967, the blowers were temporarily fitted with rented motors. A photograph of the blower equipment is shown in Figure 17.

The proximity of the aeration tank structure to the existing trickling filter required close watch of the embankment between these two structures (Figure 18). At one time when the embankment's stability was in question, shoring and backfill operations were started immediately along the south aeration tank wall. Backfilling activities were closely inspected, and on two occasions, the Contractor recompacted those areas where compaction test results were below the minimum acceptable 95 percent relative density.

Trickling Filter

The new trickling filter was the only structure constructed on engineered fill. Relative density compaction tests at this fill always resulted in values higher than the 95 percent required.

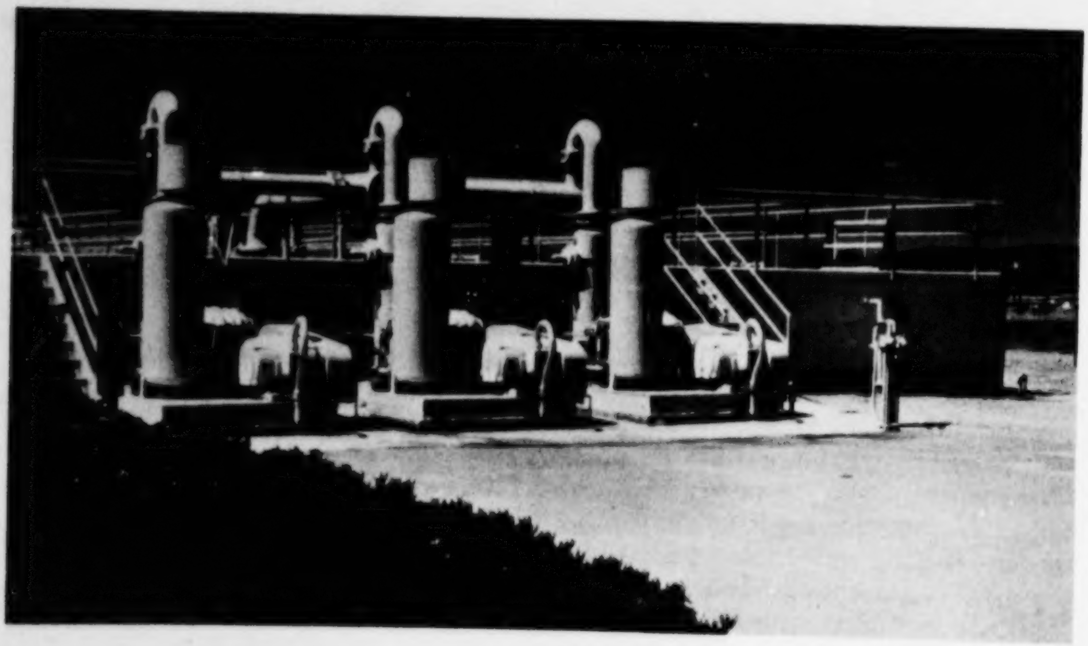


Figure 17. Aeration blower equipment. Installation proceeded behind schedule because of delivery delays.

The concrete volume in this structure was over 265 m^3 (350 yd^3). The maximum size of the aggregate for the central collection trough and pier was 3.8 cm (1.5 in.); for other portions it was 1.9 cm (0.75 in.). The floor slab was poured in two sections. The outside wall was divided into six sections based on 60-degree central angles. Alternate pie-shape sections were then poured. The inner wall holding the filter media was also divided in six sections, and alternate pie-shape sections were poured concurrently.

Underdrain installation was completed by the end of 1966, and filter media placement by the middle of January 1967. Delivery of the trickling filter distributor was behind schedule, but since this unit was not as essential to the overall process as the aeration blowers, it was believed unnecessary to air-freight the distributor arms to expedite delivery.

Secondary Sedimentation Tank

Maximum aggregate size for the concrete in which the return activated sludge pipe was embedded and for the central pier casting was 3.8 cm (1.5 in.); for the remainder of the tank, it was 1.9 cm (0.75 in.). The total quantity of concrete poured at this tank approached 420 m^3 (550 yd^3), and half of

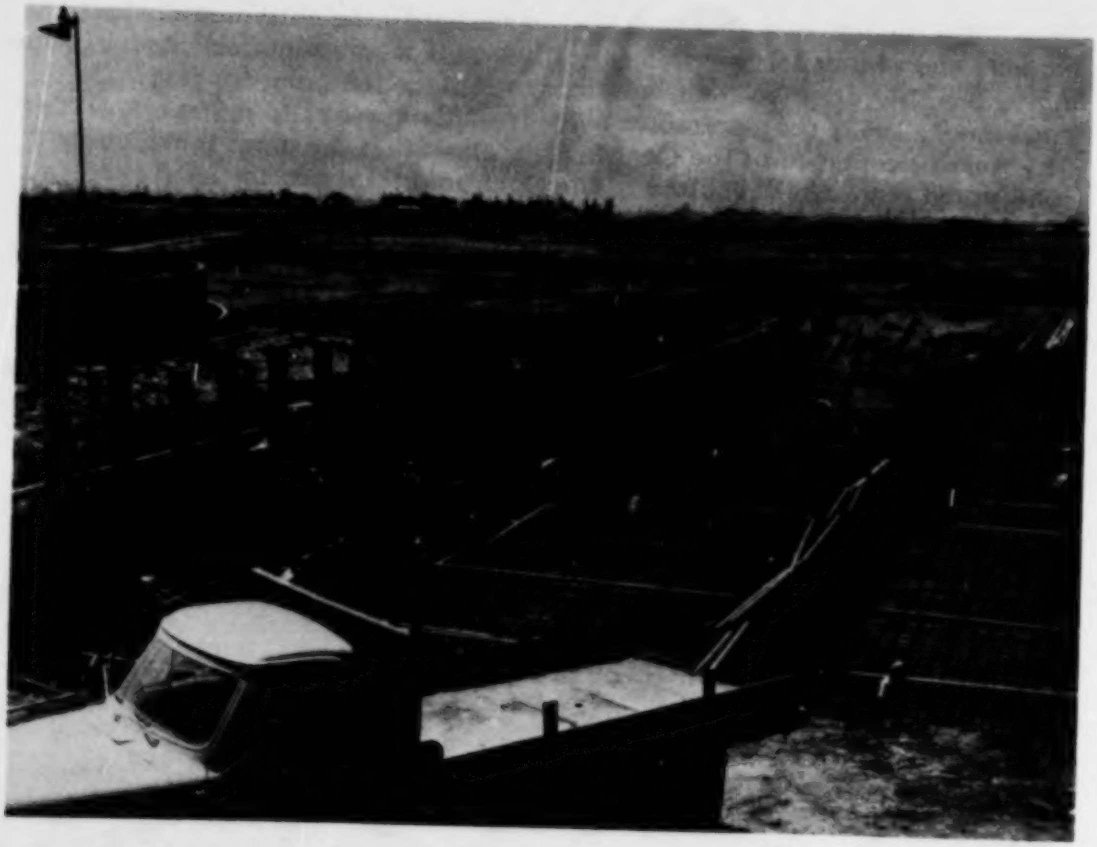


Figure 18. Aeration tank reinforcing steel placement in progress. Proximity of aeration tank excavation to the trickling filter structures caused concern over bank stability.

this total was in the floor slab. The slab was poured in four sections, and because they were not cast to final grade, a 5-cm (2-in.) grout topping was required on the structural slab.

The grout topping was placed after the mechanical equipment was installed inside the tank, and the structural steel rotating arms were utilized to spread and surface-finish the topping. Difficulties with the mechanical equipment resulted in an uneven and rough surface finish. Months later, it was noted that the grout was not bonded to the structural floor slab; additionally, several larger-than-hairline-size cracks appeared on the surface. Several meetings were held between the Contractor, the City, and Brown and Caldwell to resolve the question of responsibility for the required corrective work. The operation of this tank was essential to the activated sludge process and was necessary to meet effluent discharge requirements. Accordingly, the City directed the Contractor to replace the grout topping. On January 27, 1967, the Contractor agreed, under duress, to replace it. The City's nonacceptance of the Contractor's claim that the grout replacement constituted extra work resulted in the Contractor's action for legal redress. The suit by the Contractor against the City and its consultant was decided, years later, in favor of the City.

Equipment deliveries for this structure were not delayed, and the equipment inside the tank was assembled, welded, painted, and tested by the middle of November 1966. This equipment is shown in Figure 19. The return activated sludge pumping equipment, located outside the tank, was installed by the end of December 1966.

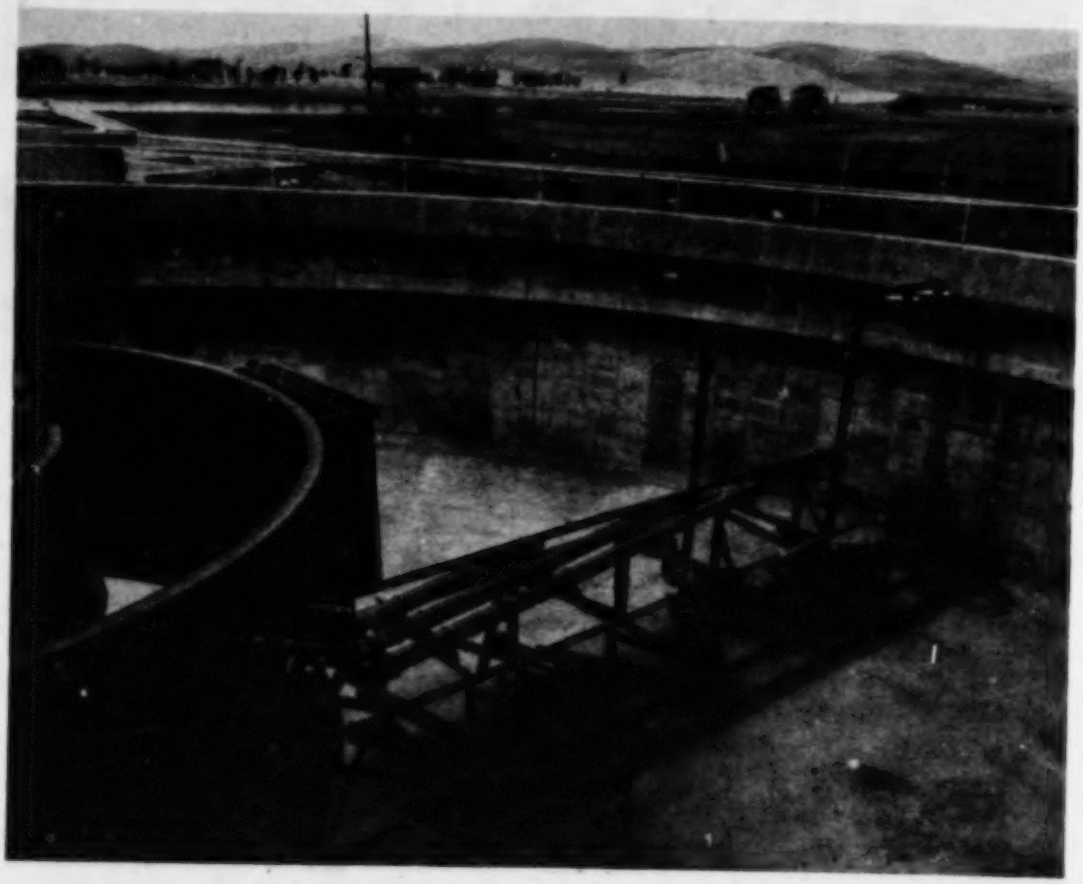


Figure 19. Secondary clarifier structure and mechanical equipment.

Chlorine Contact Tank

Expansion of the chlorine contact tank was started and completed early. An aggregate base was placed on a compacted subgrade, and the concrete was placed in two pours, one for the floor slab and the other for the walls and cross beams. Over 160 m^3 (210 yd^3) of concrete were used with a maximum aggregate size of 3.8 cm (1.5 in.). The air piping was installed by the end of September 1966, and 2 mo later the tank was placed in service.

Chlorine Storage Area

Construction activity extended from April through October 1966. Column footings and floor slabs were in place by June, but because of a delivery error, the structural frame was not available until July. The structure was erected, roofed, and drained in 1 mo and the piping and equipment were installed and painted during October (Figure 20).

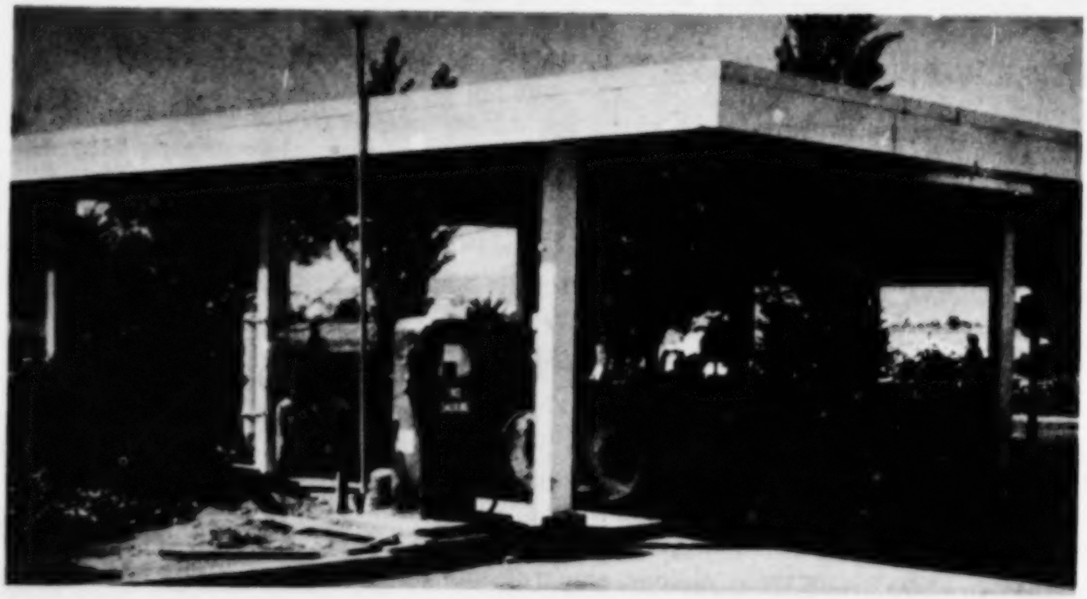


Figure 20. Chlorine storage area.

Effluent Discharge Structure

Construction activity at the effluent discharge structure was divided into two noncontinuous periods. The earlier period corresponded to the concrete work; the latter period, the installation of the water pump units and related piping and effluent sampling equipment.

Secondary Distribution Box

The construction of the secondary distribution box also result in two noncontinuous periods of activity similar to those for the effluent discharge structure. In this case,the latter period involved mostly the installation of the air-lift piping. The sluice gates inside the box were installed soon after the completion of the concrete work during the earlier period.

Filter Circulation Sump

In addition to the structural modifications to the existing filter circulation sump, construction activity involved the installation of pumping equipment for the trickling filters and aeration tanks. These pumps were in place by the end of November 1966. However, the electric motors for the aeration tank feed pumps were missing because of late deliveries. The Contractor was asked, and he agreed, to air-freight them for a mid-January 1967 delivery. Installation was accomplished soon after delivery.

Airport Irrigation Pump Station

Completion of the airport irrigation system was scheduled for June 30, 1966. Construction activities at the pump station took place during May and June, and by the end of June the pumps were installed. Electric motors had to be rented, however, to meet the deadline. The pump station was initially manually operated because the float control and panel were not installed until July. One of the motors failed in late July because of improper wiring connections. This same motor failed again 5 mo later when it shorted due to improper insulation.

Sludge Lagoons

Sludge lagoons were started and completed early during the construction period. Material excavated from the aeration tank area was used for construction of the dikes. Compaction test results averaged 95 percent relative density. The dikes were shaped and finished by early May 1966, and digested sludge was hauled in immediately. Supernatant structures and walkways were installed by December of the same year.

Holding Basins

Water and sludge had to be removed from each oxidation pond prior to conversion to holding basins. Basin 1 (oxidation pond 2) was completed first in June 1966. Later activities included demolition of the abandoned dikes and surface grading at basin 1. Dike location in basin 2 was revised during construction with new dikes being shortened and dikes scheduled to be abandoned left in service.

Outfall

The 1.1-m (42-in.) outfall from the effluent discharge structure to Arroyo Las Positas was also completed before the June 30, 1966, deadline. Changes in horizontal alignment were required to clear private property. A temporary equipment accessway through private property was necessary, and negotiations were handled directly between the Contractor and the individual landowner. This pipeline also feeds the irrigation systems at the airport and golf course.

Airport Irrigation Piping

The installation of the piping and sprinklers at the airport was started in April 1966, and the equipment ready for testing by the end of June. Leaks were discovered in the pipeline, and four additional tests were required to discover and repair all the leaks and pipeline breaks. Thereafter, the sprinklers required periodic checking. Also, the controllers for the irrigation valves were erratic in their operation and it was October before all the components were working properly.

Golf Course Irrigation Line

The 0.36-m (14-in.) line feeding the golf course irrigation storage lakes was installed during May 1966, and water to the lakes was delivered early in June. Controls for the butterfly valve were installed later.

Mixed Liquor Line

The new 0.91-m (36-in.) mixed liquor line runs from the aeration tank to the secondary distribution box and then to the secondary sedimentation tank. Two lines were installed, one for future use as shown previously in Figure 14. The lines were installed in August 1966; in December, a section next to a flexible coupling just outside the sedimentation tank structure failed. Apparently, the steel plate thickness was less than that shown in the shop drawings.

Return Activated Sludge Line

A 0.30-m (12-in.) return activated sludge line connects the aeration tank to the return activated sludge pumps. This line was installed in a common trench concurrently with the two 0.91-m (36-in.) mixed liquor lines.

Bypass Line and Manhole

The new 1.1-m (42-in.) bypass pipeline extends from the existing 1.4-m (54-in.) bypass to the new holding basin (see Figure 14 in Section 5). The bypass manhole is in the 1.4-m (54-in.) line and connects it to the secondary sedimentation tank effluent line. The new bypass line was installed between September 19 and October 25, 1966. The concrete manhole was completed a few days later. Since the 1.4-m (54-in.) line was in use (it was connected with the secondary effluent line), it was difficult to make any connection without affecting plant operation. Thus, a plant shutdown from midnight to 7:00 a.m. was scheduled. However, three such shutdowns were required before the connections were successfully made.

Aeration Tank Drain and Scum Line

A 25-cm (10-in.) tank drain and 15-cm (6-in.) scum line extended from the aeration tank to the inlet works. In the vicinity of the inlet works, the new lines ran alongside an existing supernatant line which had been leaking for some time; consequently, the ground was moist.

A vitrified clay pipeline for raw sludge was also broken during excavation in this area. The broken line was repaired, but a leak developed in the repaired area at a later date. Because of the nature of the affected pipelines, it was necessary to shut down the plant to effect the repairs. The excessive water from the leaks and breaks also affected the size of the excavation, and the hole grew bigger and bigger. Pea gravel fill material was imported to protect the inlet works. All this activity was considered extra work by the Contractor, and it took several meetings to reach a compromise on the amount of the extra cost.

PROGRESS PAYMENTS

In addition to the daily reports and related information mentioned previously, the requests for monthly progress payments and extra work also outline the Contractor's progress throughout the construction period. The rate of progress payments, based on the Contractor's bid breakdown, is shown graphically in Figure 21 for major construction items. The payment percentage is related to the total cost of the item. Examination of Figure 21 reveals that the payments went beyond the time originally allowed for completion. The cumulative progress payments are shown in Figure 22. Most of the work had been accomplished by May 1967 when the City partially accepted the improvements. The big jump in payments during 1967 was the release of money withheld by the City for uncompleted construction items. Although the construction work was substantially complete, some of the finishing touches

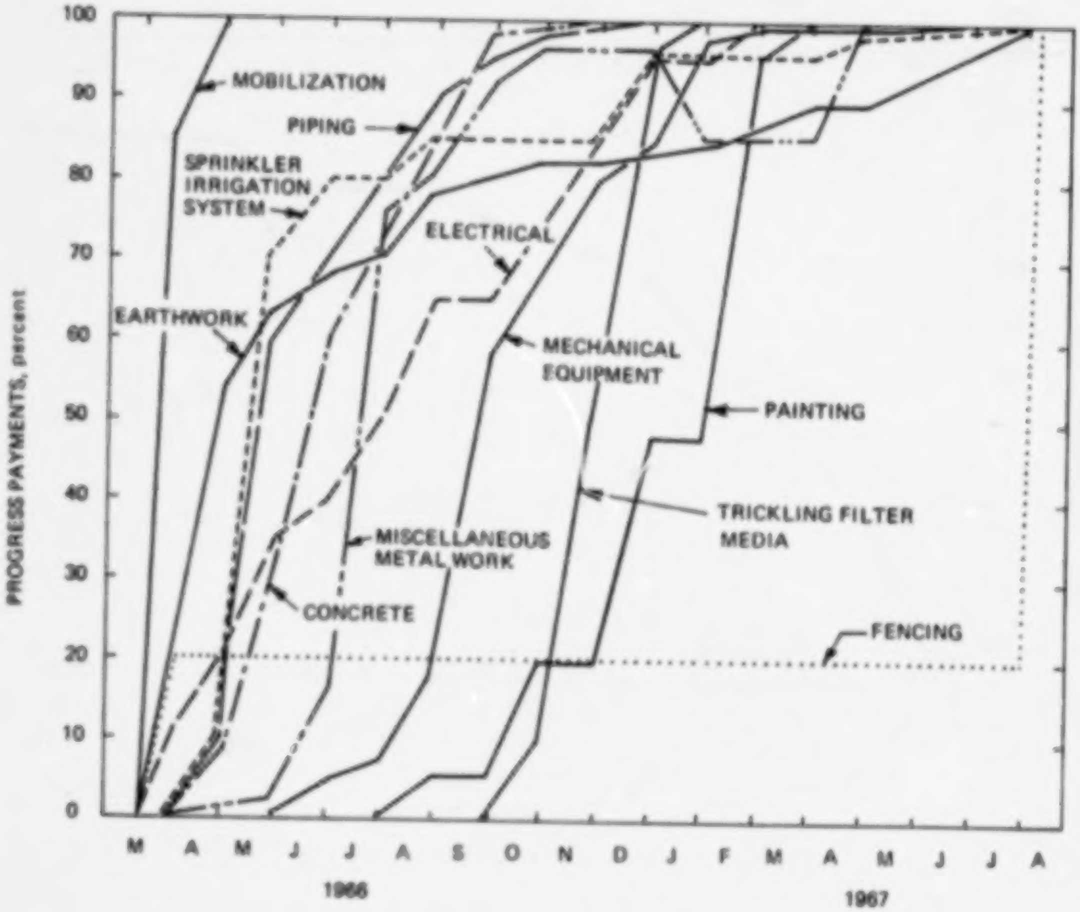


Figure 21. Progress payment rates for major construction items.

had not been completed and this affected the amount of money retained by the City. Progress payments extend throughout 1967 because of the difficulty in reaching an agreement on the extent of the extra work orders.

Extra work orders 1 through 33, in the amount of $8,101.40, were finally paid for on December 29, 1967. The overall cost of the change orders is a small percentage of the construction cost. A summary of the total construction cost is shown in Table 8.

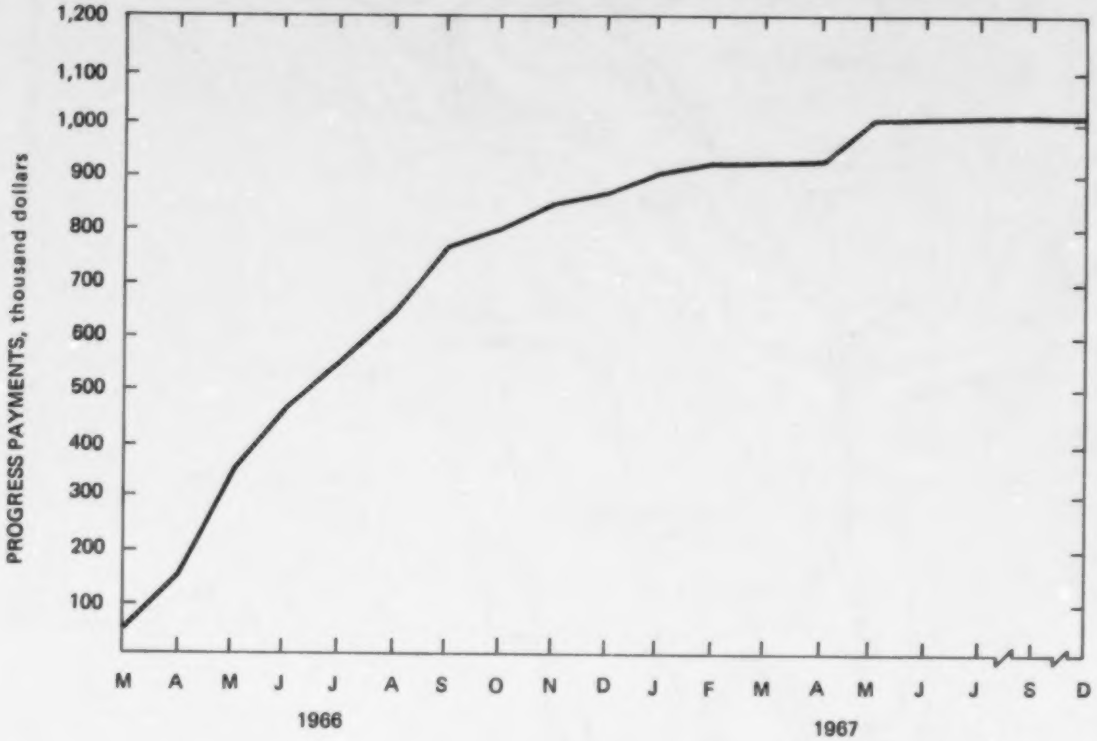


Figure 22. Cumulative progress payments.

ACCIDENTS

No fatal accidents occurred during the construction period. Only three accidents were recorded at the job site: (1) a plant operator received a cut wrist and lacerated leg when he fell through a loose grating into a pipe channel near the primary sedimentation tank; (2) a Contractor superintendent's right foot was fractured by a falling concrete hose caused by a broken connection; (3) an operator hurt his foot when it was caught in a V-pulley. Another accident which took place near the job site was considered a traffic accident as it occurred at the entrance to the job site, but outside the gates.

COMPLETION

The contract documents established the date of completion as of January 10, 1967. It was late March, however, before startup of the activated sludge process, the most important plant component, was begun (see

TABLE 8. TOTAL CONSTRUCTION COST

Item	Cost, dollars
Bid price	1,057,800.00
Add for equipment charge	10,000.00
	1,067,800.00
Deduct contingency fund	20,000.00
	1,047,800.00
Change Orders 1-33	8,101.40
	1,055,901.40
Total payment	1,055,901.40

Section 7). As mentioned previously, the City opted for no liquidated damages if the new facilities were in operation in time to meet the regulatory agency requirements. Although the Contractor's time was not officially extended, the position adopted by both parties was that the project would be finished as soon as possible. Inability to agree on change orders and the extent of unfinished work appears to have affected completion and acceptance of the project. The City Council agreed on May 9, 1967, to partially accept the improvements, and the notice of completion was issued by the City Engineer on August 14, 1967.

SECTION 7

PLANT STARTUP AND INITIAL OPERATING PERIOD

During construction of the upgraded Livermore Water Reclamation Plant, the City was prohibited by the Regional Water Quality Control Board from discharging inadequately treated effluent to the surface waters of Arroyo Las Positas. To meet the discharge prohibition, the City utilized portable aluminum piping and a temporary pumping system to spray irrigate effluent onto 60 ha (150 ac) of leased lands and the nearby municipal golf course. Because of the expense and limited capacity of this interim disposal method, it was imperative that, once completed, the plant be fully operational and producing a satisfactory effluent as soon as possible.

This section describes the startup and initial operating period of the plant. Procedures used in bringing the activated sludge system on line, initial operational problems, and performance data from March through December 1967 are the main topics discussed.

A principal problem discussed below was the inability to consistently nitrify in the activated sludge unit. After several months of attempting various solutions, it was determined that an inadequate air supply was the cause. Utilizing the third (standby) blower for portions of the day resulted in significant improvement, but as discussed in Section 8, the problem has recurred over the years each time a blower has been shut down for repairs.

PREPARATION FOR STARTUP

In December 1966, approximately 3 mo before the estimated project completion date, the City increased the plant staff from five to nine by hiring two more operators, a laboratory technician, and a maintenance mechanic. A training program was begun which included familiarizing all personnel with the operation of existing portions of the plant. One-hour-per-day training sessions on the new portions of the plant lasted approximately 2 mo until the construction was completed. The existing employees trained the new personnel in general operation, preventive maintenance, and laboratory procedures for the original plant.

Materials used for training included the preliminary Operation and Maintenance Manual prepared by Brown and Caldwell, an operator training manual prepared by the California Water Pollution Control Association, and manuals prepared by various state agencies and universities. A special effort was made to find training guides oriented toward nitrification in the activated sludge process, but the only information found was in Journal Water Pollution Control Federation articles. This again indicates the lack of widespread knowledge on nitrification which existed at the time.

In addition to utilizing available literature, discussions were held between the plant staff and design engineers from Brown and Caldwell. These meetings yielded information and suggestions which were instrumental in developing an understanding of the plant's capabilities.

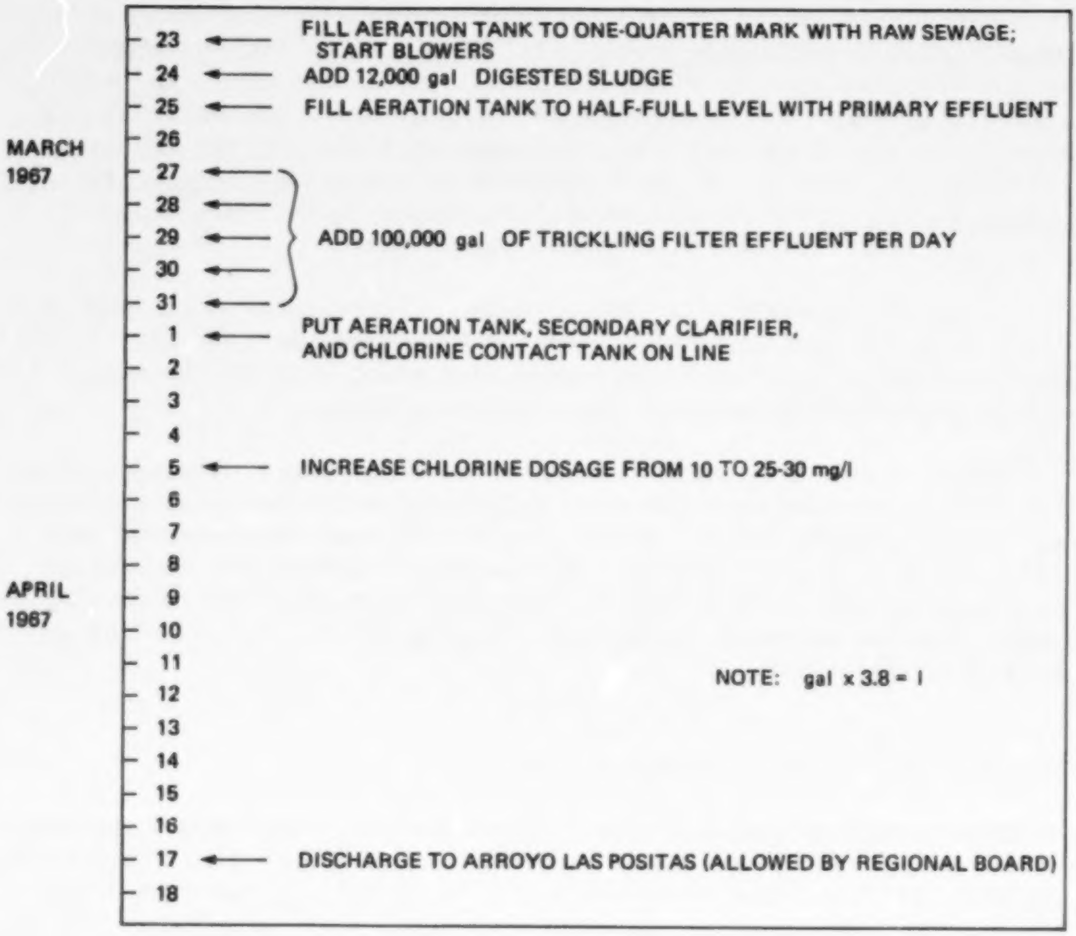
Before startup all pumps, valves, blowers, collectors, flowmeters, air diffusers, and piping were inspected by City inspectors and plant personnel and were thoroughly tested. All dirt, rocks, and trash were removed from lines, pumps, valves, and tanks, and systematic testing was made of all electrical controls. The Contractor confirmed the startup date 10 days beforehand. After the activated sludge unit was ready for use, a final check was made of all equipment.

STARTUP OF ACTIVATED SLUDGE SYSTEM

In conjunction with the design engineering firm, a plan was formulated for making the activated sludge system operational (Figure 23). The first step consisted of filling the aeration tank one-quarter full with raw sewage and starting one of the blowers. After a 24-hr period, 45 m^3 (12,000 gal) of anaerobically digested sludge were added to the aeration tank. The tank was half filled with raw sewage and kept under aeration for 2 days. At that time, additional wastewater from the trickling filter was added periodically at a rate which filled the tank in 5 more days.

Dissolved oxygen (DO), mixed liquor suspended solids (MLSS), and sludge settleability (sludge volume index or SVI) were analyzed at 4-hr intervals and DO maintained at 2.5-3.5 mg/l. The MLSS level averaged 1,300 mg/l during the 5-day period, and the 30-min settled volume decreased from 600 ml/l to 250 ml/l by the fifth day. A well-defined floc was noted on the second day.

The total flow from the trickling filter was added to the aeration tank on the tenth day, and operation of the secondary clarifier was begun. The suggested loading rate was 15-20 kg BOD_5/kg MLSS/day; using an estimated filter effluent BOD_5 of 120 mg/l, an MLSS level of 1,400 mg/l was required.


MARCH 1967
APRIL 1967
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18
FILL AERATION TANK TO ONE-QUARTER MARK WITH RAW SEWAGE; START BLOWERS
ADD 12,000 gal DIGESTED SLUDGE
FILL AERATION TANK TO HALF-FULL LEVEL WITH PRIMARY EFFLUENT
ADD 100,000 gal OF TRICKLING FILTER EFFLUENT PER DAY
PUT AERATION TANK, SECONDARY CLARIFIER, AND CHLORINE CONTACT TANK ON LINE
INCREASE CHLORINE DOSAGE FROM 10 TO 25-30 mg/l
NOTE: gal x 3.8 = l
DISCHARGE TO ARROYO LAS POSITAS (ALLOWED BY REGIONAL BOARD)

Figure 23. Startup sequence for activated sludge process.

Effluent from the secondary clarifier contained 20-30 mg/l suspended solids. Suspended solids levels in the return activated sludge were 0.4-0.5 percent. Secondary effluent was chlorinated at a dosage of 10 mg/l. Combined chlorine residuals of 2-3 mg/l were measured at a contact time of 120-150 min. On the second day of full aeration tank operation, a clear effluent was monitored with suspended solids concentrations ranging from 15-20 mg/l. A BOD_5 value from a settled aeration tank mixed liquor sample taken 5 days previously measured 20 mg/l.

Total coliform organism MPN's, however, ranged from 50 to 250/100 ml, far greater than the 5-day median discharge requirement of 5.0/100 ml. To correct this, the chlorine dosage was increased to 25-30 mg/l, resulting in residuals of 10-20 mg/l. On the third day of operation, coliform counts were ranging from 2.2 to 5.0 MPN/100 ml, and after 5 consecutive days with counts of 5.0 MPN/100 ml or less, the Regional Board allowed surface discharge of effluent, permitting an end to the temporary land irrigation system. The operating aeration tank is shown in Figure 24.

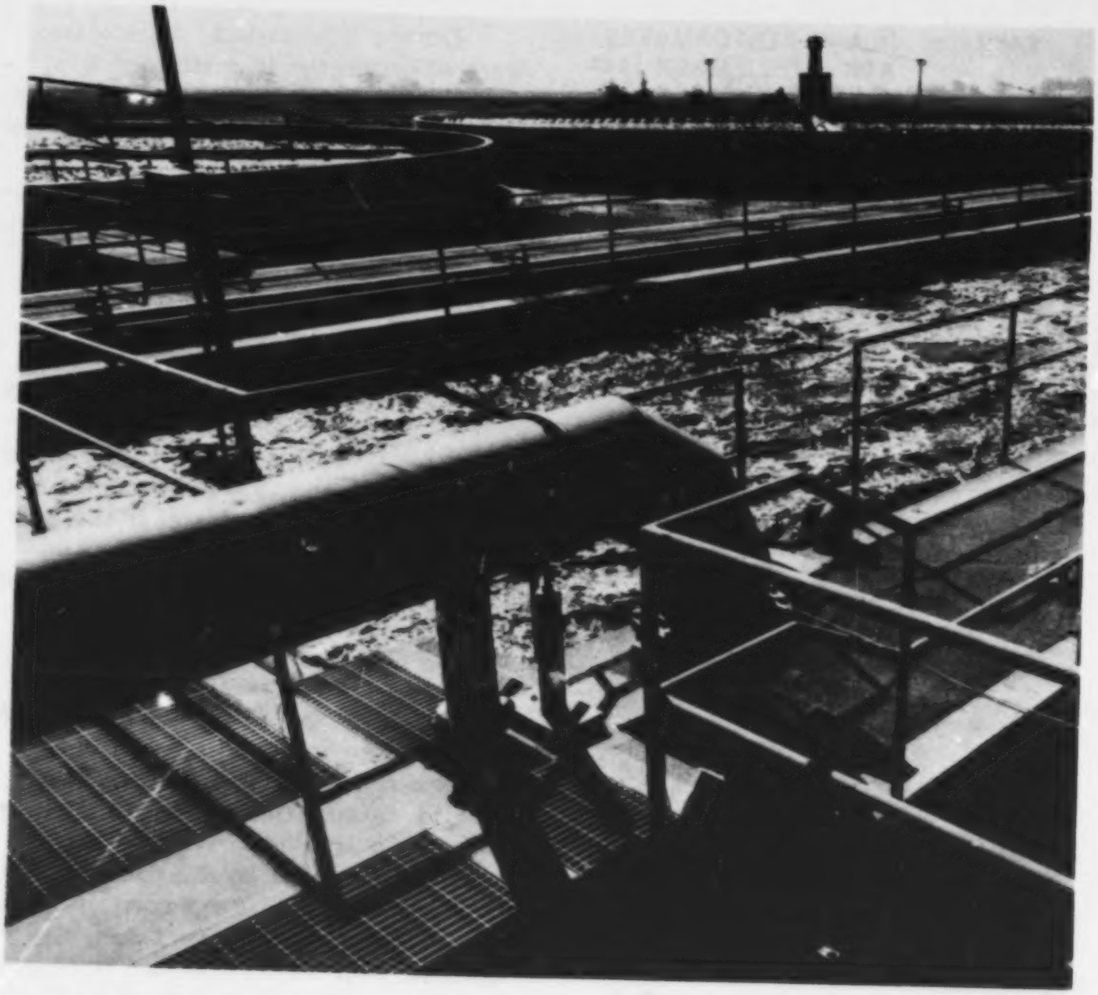

Figure 24. Activated sludge aeration tank in operation. Tank was placed on-line on April 1, 1967. Surface discharge was allowed 17 days later. Roughing trickling filters are in background.

INITIAL OPERATIONAL PERIOD

The performance of the Livermore plant from April through December 1967, when the new activated sludge unit was operating, is summarized in Table 9. Although excellent BOD_5 and suspended solids removals were obtained, analysis of final effluent ammonia nitrogen concentrations indicated that complete nitrification was not occurring. This persisted until early 1968 while several remedies were tried, none of them successful.

TABLE 9. PLANT PERFORMANCE: APRIL-DECEMBER 1967

Parameter	Value
Flow, mgd[a]	3.0(est.)
BOD_5	
Influent, mg/l	190
Effluent, mg/l	15
Removal, percent	93
Suspended solids	
Influent, mg/l	220
Effluent, mg/l	11
Removal, percent	95
pH	
Influent	7.6
Effluent	7.6
Effluent analyses	
Grease, mg/l	5.4
MBAS, mg/l	0.52
Ammonia-nitrogen, mg/l	
LWRP[b]	4.4
EQA[c]	11
Nitrite-nitrogen, mg/l	
LWRP[b]	0.35
EQA[c]	0.23
Nitrate-nitrogen, mg/l	
LWRP[b]	7.9
EQA[c]	4.7
Chlorine residual, mg/l	7.5
Coliform concentration, MPN/100 ml	
Median	2.1
90th percentile	25

[a] mgd x 0.044 = m^3/sec

[b] Measured by plant staff.

[c] Measured by Environmental Quality Analysts, Inc.

During this period, difficulties were encountered in maintaining a uniform DO level in the aeration tank. The level sought was 2.5-3.5 mg/l at the effluent overflow weir. Use of fixed-speed blowers required that adjustments be made by turning a blower on or off. This resulted in constant checking and adjusting, with DO levels fluctuating greatly. An additional problem was the necessity of making DO analyses in the laboratory, a time-consuming operation since measurements were being taken along the length of the tank. Installation of a DO analyzer-recorder near the aeration tank overflow weir in July 1968 allowed more immediate monitoring of DO levels.

Although adequate disinfection was obtained by utilizing high chlorine dosages, complete nitrification was not attained until an increase in air flow to the aeration tank was tried in early 1968 (Figure 25). This was accomplished by using the third blower on a regular basis for a portion of the day. The target DO level near the effluent weir was increased from about 2.5-3.5 mg/l to 4.5-5.5 mg/l.

An important aspect of Figure 25 is whether partial nitrification was ocurring continuously from June 1967 through January 1968 or whether nitrification took place only during low loading periods of the day with major ammonia bleedthrough occurring during peak loading periods. The values shown in Figure 25 are based on monthly averages of 24-hr composites taken once a week. From the occurrence of some nitrate in the effluent, it can be assumed that some nitrification was taking place. It is probable that this was during low loading periods of the day when the quantity of air supplied was sufficient to keep the DO level high enough for the nitrifying reactions to occur.

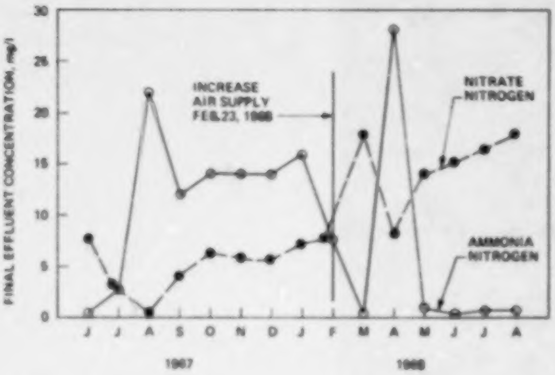

Figure 25. Ammonia nitrogen removal and blower operation.

The oxygen demand imposed by ammonia nitrogen in a nitrifying system is quite rapid, and a loading increase can cause rapid DO depletion with subsequent loss of nitrification. As the DO level increases, nitrification will start again. When nitrogen analyses are made on 24-hr composite samples, it often appears that low-level or partial nitrification is occurring continuously when, in fact, the nitrification level has varied greatly over the 24-hr period.

The reason for the difficulty in solving the problem was the scarcity of available information on nitrification in full-scale plant operation. The lack of adequate knowledge for design was discussed in Section 5, and the difficulties encountered in obtaining operator training information were noted above. More widespread use of nitrification in wastewater treatment since 1967 has resulted in better designs and the availability of better training materials. Factors pertinent to nitrification in coupled trickling filter-activated sludge systems will be discussed in Section 10.

Another operational problem which developed early was the occurrence of denitrification in the secondary clarifier. It was found that the sludge blanket has to be maintained at a depth of less than 1/3 m (1 ft) after the collector has passed by; depths greater than this result in denitrification with clumps of sludge being buoyed to the surface by nitrogen gas. A photoelectric cell to detect sludge blanket height was built by the plant staff (Figure 26). It is operated by lowering in into the clarifier until a decrease in light transmission is detected by the cell (Figure 27). The length of line under water indicates the depth to the beginning of the concentrated sludge blanket.

The unusually high chlorine dosages of 20-30 mg/l also created problems in adding chlorine and measuring residuals because the equipment was designed for normal, lower dosages. The problems eventually necessitated

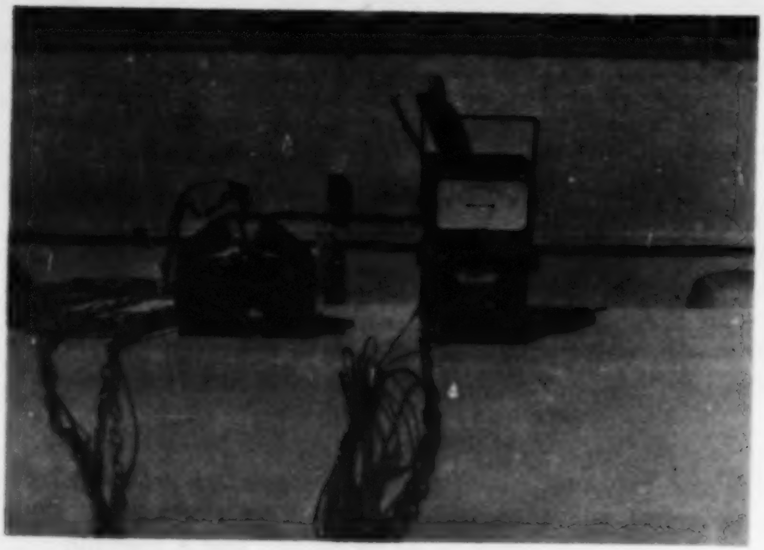

Figure 26. Photoelectric probe (on left) for sensing sludge blanket height. Reading is taken on meter (on right).

Figure 27. Photoelectric probe being lowered into secondary clarifier. This measurement is made after sludge withdrawal arms have passed. Sludge blanket is kept at 1/3 m (1 ft).

abandonment of the chlorine residual analyzers and resulted in development of a control method utilizing pH as the control parameter. This method will be discussed further in Section 8.

Increased solids production also caused immediate problems. Difficulties were encountered in maintaining a high solids content in the raw sludge pumped to the anaerobic digesters. This adversely affected volatile matter reduction in the two mixed digesters and caused subsequent sludge lagoon odor problems about 2 mo after startup of the activated sludge unit. These problems have never been fully solved, although water depth in the lagoons is now maintained at the 3-m (10-ft) maximum depth, instead of the 1-m (3-ft) depth originally used, and chemical odor control equipment has been installed. Solids handling data and associated problems are discussed more fully in the following section.

SECTION 8

PLANT OPERATION AND PERFORMANCE

Since its completion in early 1967, the Livermore Water Reclamation Plant (Figures 28 and 29) has been able to consistently produce a high quality effluent from the coupled trickling filter-activated sludge process. Because of the advanced nature of the treatment process and the stringency of the discharge requirements, operation and performance records for the 0.22-m^3/sec (5-mgd) plant are unusually complete. They allow a complete discussion of operating procedures and plant performance for a full 7 yr of record from 1968 through 1974.

Although the overall performance record is good, some operational problems have occurred. In most cases, procedural changes or equipment modifications have resulted in improvement. Problems which have occurred and their possible remedies are important in the present context in order to give guidance to designers of plants which might be constructed in the future. Additionally, an evaluation of plant performance, coupled with recently developed knowledge of nitrification kinetics, can allow more precise, rational design of future plants with less reliance placed on empirical approaches.

The purpose of this section is to discuss and evaluate the operation and performance of the Livermore plant since its initial period of operation in 1967. Operational problems have included removal of solids from the primary sedimentation tanks, a long-term breakdown of one of the two trickling filters in 1968 and 1969, difficulties with the automatic control system for the filter circulation sump, an undersized aeration air supply to the aeration tank, a high secondary clarifier overflow rate, accumulation of solids in the chlorine contact tank, and sludge lagoon odors. Performance of the plant has been extraordinarily good because of lower than expected loadings, a well-trained staff, and the existence of emergency holding ponds which allow shutdown of portions of the plant without bypassing of inadequately treated wastewater to the receiving waters.

As an introduction to this section, a description of sampling methods and locations is given below. Following this and preceding detailed discussions of individual plant components is a brief summary of overall plant performance. Finally, a critical evaluation of plant performance compares operating

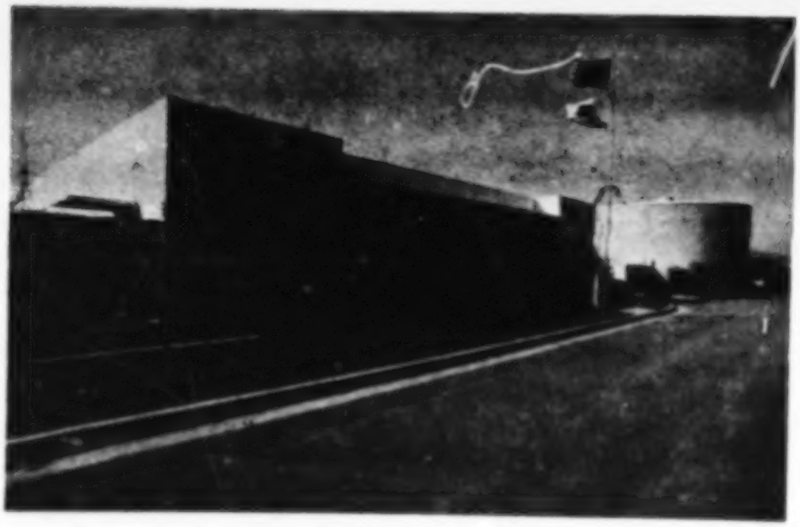

Figure 28. Operations building, Livermore Water Reclamation Plant.

Figure 29. Livermore plant control room.

data with predicted values and summarizes the weaknesses and strengths of the Livermore Water Reclamation Plant. Information from this section is drawn on extensively in the discussion of general design considerations presented in Section 10.

SAMPLING METHODS AND LOCATIONS

Sampling of the wastewater stream is undertaken at five locations: raw wastewater, primary effluent, trickling filter effluent, secondary effluent, and final (or chlorine contact tank) effluent. Two major changes in the sampling program during the period of interest require mention. During 1969 and earlier,no samples were taken of effluent from the secondary clarifier. Final effluent samples reflected settling of particulate matter in the chlorine tank and possible changes induced by the high doses of chlorine required for disinfection. Beginning in 1970, sampling of secondary clarifier effluent was incorporated in the regular sampling program.

The second change involved raw wastewater sampling. During 1969 and earlier,raw wastewater was sampled by hand from the raw wastewater sump following the metering flumes. An automatic sampler was installed in 1970, but the high level of turbulence in the sump occasionally allowed air to enter the sample line and created sampling difficulties. The raw wastewater sampling point was then moved to the preaeration and primary sedimentation tank influent channel following the raw sewage pumps. This resulted in improved operation of the automatic sampler but has had an adverse effect on measured raw wastewater characteristics because the activated sludge waste mixed liquor and sludge lagoon supernatant are recycled to the headworks ahead of the new sampling point. Review of operating records indicates that an increase in raw wastewater BOD_5 and suspended solids concentrations of 30 to 50 mg/l has resulted from this change in sampling point location. This should be kept in mind when evaluating primary treatment removals and overall plant performance during and after 1970.

Sampling methods and main-stream sampling locations are presented in Table 10. The constituents analyzed by the plant staff (Figures 30 and 31) at these locations,along with the frequency of analysis,are listed in Table 11. In addition to the constituents shown in Table 11, additional analyses of plant effluent characteristics are made by an outside laboratory and include measurement of a wide range of chemical constituents, including heavy metals. The full list of these constituents is given in Appendix D.

In addition to analyses of the main waste stream, plant records are maintained for other operating parameters, principally aeration tank mixed liquor characteristics and solids handling and treatment data. A list of these data is also given in Appendix D.

TABLE 10. PRINCIPAL SAMPLING METHODS AND LOCATIONS

Point in flow stream	Location of sampling point	Sampling method[a]
Raw wastewater 1969 and earlier	Raw wastewater sump	24-hour, flow-proportional composite; hand-sampled hourly
1970 and later	Preaeration tank influent channel	24-hour, flow-proportional composite; automatic sampler with refrigerated compartment; sampled hourly
Primary effluent	Primary clarifier effluent troughs	24-hour, flow-proportional composite; hand-sampled hourly
Trickling filter effluent	Aeration tank influent over flow channel (after 1973: at spigot tap installed on discharge side of one of the aeration tank feed pumps)	24-hour, flow-proportional composite; hand-sampled hourly
Secondary effluent (during and after 1970)	Secondary clarifier effluent channel	24-hour, flow-proportional composite; hand-sampled hourly
Final effluent	Chlorine contact tank effluent line	24-hour, flow-proportional composite; automatic sampler with refrigerated compartment; sampled hourly

[a] For once-a-week composites. Some parameters may be monitored on a daily basis.

An important limitation of the Livermore data is the absence of organic nitrogen measurements ahead of the final effluent. In plants designed for nitrification, organic nitrogen can be transformed to ammonia during biological treatment and subsequently nitrified. Thus, the sum of the ammonia nitrogen and organic nitrogen (total Kjëldahl nitrogen or TKN) is often used to compute nitrification efficiencies. Raw domestic wastewater organic nitrogen concentrations will usually be 40 to 70 percent of the ammonia nitrogen concentration.

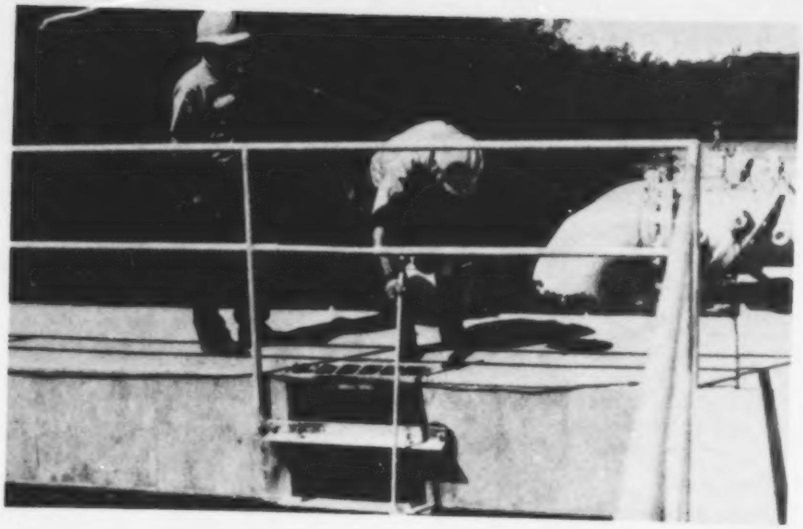

Figure 30. Demonstration of manual sampling technique. Primary effluent is hand-sampled at the primary clarifier effluent troughs.

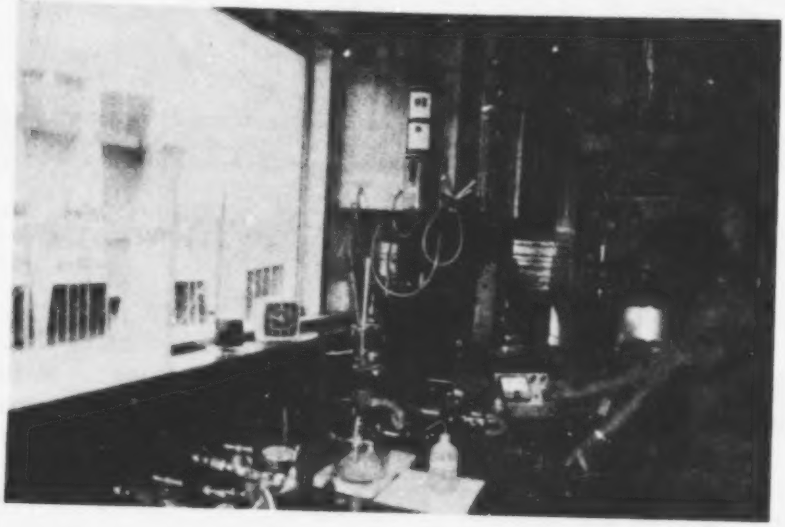

Figure 31. Plant analytical laboratory. Wastewater and sludge analyses are performed here.

TABLE 11. WASTEWATER CHARACTERISTICS ANALYZED AT PRINCIPAL SAMPLING POINTS

Parameter	Raw sewage[a]	Primary effluent	Trickling filter effluent	Secondary effluent[b]	Plant effluent
BOD_5, mg/l	W[c]	W	W	W	W
Suspended solids, mg/l	W	W	W	W	W
Volatile suspended solids, mg/l	W	W	W[d](P)	W	W
Dissolved oxygen, mg/l				D	D
Settleable solids, ml/l	W	W	W[d](P)	W	W
Total solids, mg/l	W	W	W[d](P)		W
pH	W	W		W	W
Grease, mg/l					W
MBAS, mg/l					W
Ammonia nitrogen, mg/l		W(P)	W(P)	W(P)	W(P)
Nitrite nitrogen, mg/l				W(P)	W(P)
Nitrate nitrogen, mg/l				W(P)	W(P)
Turbidity, JTU					D(P)
Temperature, C					D(P)
Total dissolved solids, mg/l	W		W[d](P)		W
Fixed dissolved solids, mg/l	W		W[d](P)		W
Specific conductivity, um	W				W
Chlorides, mg/l	W				W

[a] During and after 1970, raw sewage samples contain waste mixed liquor and sludge lagoon supernatant returned to headworks.

[b] Only for 1970 and later.

[c] W = weekly, D = daily, (P) = partial record

[d] Not recorded on monthly logs.

PLANT PERFORMANCE SUMMARY

A brief summary of plant performance over the 7-yr period from 1968 through 1974 is presented in Table 12,which provides an overview of plant performance prior to detailed discussions of specific plant components. Wastewater flowing to the Livermore plant is primarily domestic and commercial in origin as little industrial activity exists in the area. This results in a consistent influent quality which has aided in the production of a high quality effluent.

Flows have remained fairly constant over the period, ranging from 0.16 m^3/sec (3.7 mgd) in 1968 to 0.20 m^3/sec (4.6 mgd) in 1972. It has been postulated exfiltration problems are in part responsible for the flow decreases in 1973 and 1974, but this has not been substantiated.

The effect of recycled waste mixed liquor and sludge lagoon supernatant is reflected by the increase in raw wastewater BOD_5 and suspended solids

TABLE 12. PERFORMANCE SUMMARY

Parameter	1968	1969	1970	1971	1972	1973	1974	Overall average
Flow, mgd[a]	3.7	4.0	3.8	3.9	4.6	4.1	4.1	4.0
Raw Influent[b]								
BOD_5, mg/l	180	180	240	210	230	220	210	210
Suspended solids, mg/l	210	180	230	230	260	240	250	230
Primary Effluent								
BOD_5								
Concentration, mg/l	140	130	140	130	130	120	130	130
Removal, percent	22	28	42	38	43	45	38	38
Suspended solids								
Concentration, mg/l	95	76	83	79	110	80	79	86
Removal, percent	55	58	64	66	58	67	68	63
Ammonia nitrogen, mg/l	-	-	43	41	36	52	41	43
Secondary Effluent								
BOD_5								
Concentration, mg/l	-	-	14	10	15	16	19	15
Removal, percent[c]	-	-	90	92	88	87	85	88
Suspended solids								
Concentration, mg/l	-	-	19	22	40	28	24	27
Removal, percent[c]	-	-	77	72	64	65	70	69
Ammonia nitrogen								
Concentration, mg/l	-	-	0.95	1.7	1.0	1.5	1.7	1.4
Removal, percent[c]	-	-	98	96	97	97	96	97
Final Effluent								
BOD_5, mg/l	9.6	6.6	8.1	7.3	8.1	9.4	11	8.6
Suspended solids, mg/l	9.5	12	12	13	22	28	18	16
Volatile suspended solids, mg/l	7	8	9	4	14	18	16	11
Ammonia nitrogen, mg/l	4.9	0.51	<0.11	<0.2	0.3	1.2	0.73	1.1
Organic nitrogen, mg/l	1.8	1.3	1.0	2.2	2.7	2.6	1.5	1.9
Nitrate nitrogen, mg/l	14	22	22	22	22	23	25	21
Total nitrogen, mg/l	21	24	23	24	25	27	27	24
Total solids, mg/l	1,090	800	780	780	760	860	800	840
Total dissolved solids, mg/l	1,050	770	760	770	740	840	780	820
Fixed dissolved solids, mg/l	800	510	490	530	480	540	540	560
Grease, mg/l	4.0	2.6	2.5	2.3	5.1	5.2	7.1	4.1
MBAS, mg/l	0.39	0.15	0.07	0.04	0.08	0.06	0.17	0.14
pH	7.5	7.3	7.0	6.7	6.9	6.9	6.7	7.0
Alkalinity, mg/l	110	85	74	68	52	58	32	68
Hardness, mg/l	280	210	200	200	190	210	190	210
Total coliforms, MPN/100 ml[e]	2.1	2.1	2.1	2.1	2.1	2.1	<2.1	2.1
Overall Removals, percent								
BOD_5[d]	95	96	97	97	96	96	95	96
Suspended solids[d]	95	93	95	94	92	88	93	93
Ammonia nitrogen[c]	-	-	>99	>99	99	98	98	99

[a] mgd x 0.044 = m^3/sec

[b] Includes waste mixed liquor and lagoon supernatant recycled to headworks during and after 1970

[c] Based on primary effluent

[d] Based on raw influent

[e] Median

values after the change in sampling points at the beginning of 1970. This effect does not appear to extend beyond the raw wastewater values, however; the data indicate that all of the recycled solids are removed by primary sedimentation.

Removal of suspended solids and BOD_5 in the chlorine contact tank is indicated in Table 12. Although this provides a slightly better quality effluent, it also necessitates periodic cleaning of the tank, which requires plant shutdown. This problem will be discussed in more detail later.

An informative method of illustrating the consistency of plant performance involves plotting effluent quality in probability form; effluent concentration (of BOD_5, for example) is compared with the percentage of time during which that value is exceeded. Probability curves for secondary and final effluent BOD_5 concentrations in 1973 are plotted in Figure 32. Median values are 14 and 9 mg/l, respectively. Ninetieth (90th) percentile values, those concentrations which are exceeded only 10 percent of the time, are 26 and 15 mg/l, respectively. Similar results were achieved in other years.

High secondary effluent ammonia nitrogen concentrations affect the disinfection efficiency of the plant by preventing formation of free chlorine residuals. Ammonia nitrogen probability curves for three years, 1970 through 1972, are plotted in Figure 33. Median values range from 0.82 to 0.99 mg/l, and 90th percentile values fall between 1.3 and 1.9 mg/l. One value over 10 mg/l was recorded during the 3-yr period and resulted from temporary shutdown of one of the three blowers and consequent loss of full nitrification.

Final effluent coliform concentration (MPN/100 ml) probability curves are shown for the same 3-yr period in Figure 34. The median, 90th percentile, and 95th percentile values are approximately 2, 15, and 40 MPN/100 ml, respectively.

Data presented above and in subsequent discussions have been chosen for their illustrative value and, except where otherwise noted, are representative of general plant operation. Often, to reduce the amount of material presented in the text, data from only a portion of the full 7-yr period are used in graphs or tables. In addition, some data are completely omitted from the text on the basis that their general interest is minimal. Appendix E comprises monthly and yearly averages of most of the operating and performance parameters normally monitored at Livermore. The reader is referred to that appendix for more complete information on plant operation and performance.

OPERATION OF PLANT COMPONENTS

To provide a more detailed discussion of plant operations and performance, it is convenient to divide the facility into several units or components and

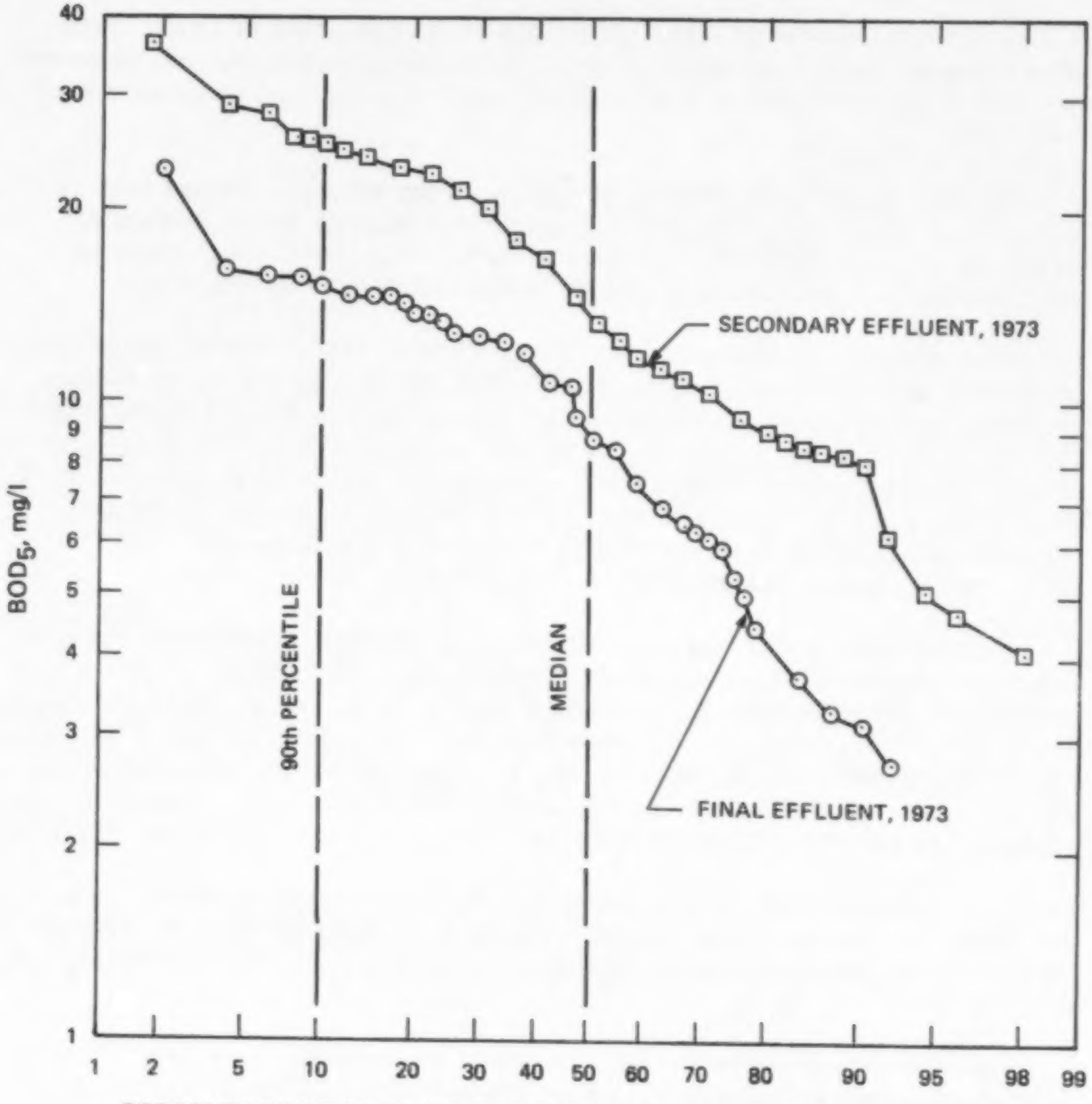

Figure 32. Probability curves for secondary and final effluent BOD_5.

cover them individually. For purposes of this presentation, the Livermore plant has been divided into eight components: (1) flows and flow metering, (2) preliminary and primary treatment, (3) trickling filtration, (4) activated

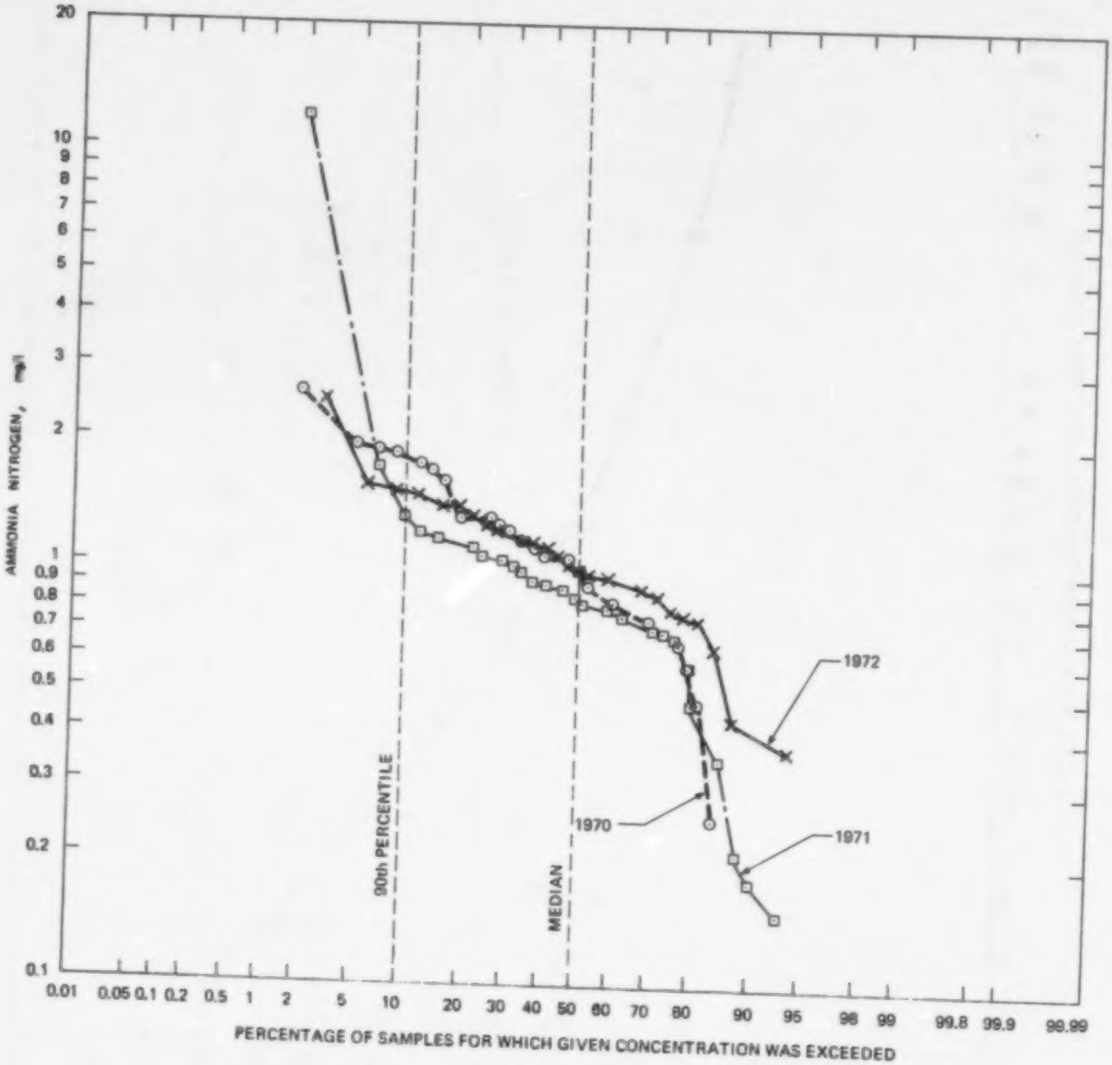

Figure 33. Probability curves for secondary effluent ammonia nitrogen.

sludge, (5) secondary clarification, (6) chlorination, (7) solids handling, and (8) reclamation. For each component, a discussion is presented covering its operational characteristics and any changes in operating procedures developed by the plant staff in response to problems encountered. Performance data are graphically or tabularly presented, and, where relevant, actual performance is compared with predicted performance. Subsequent to

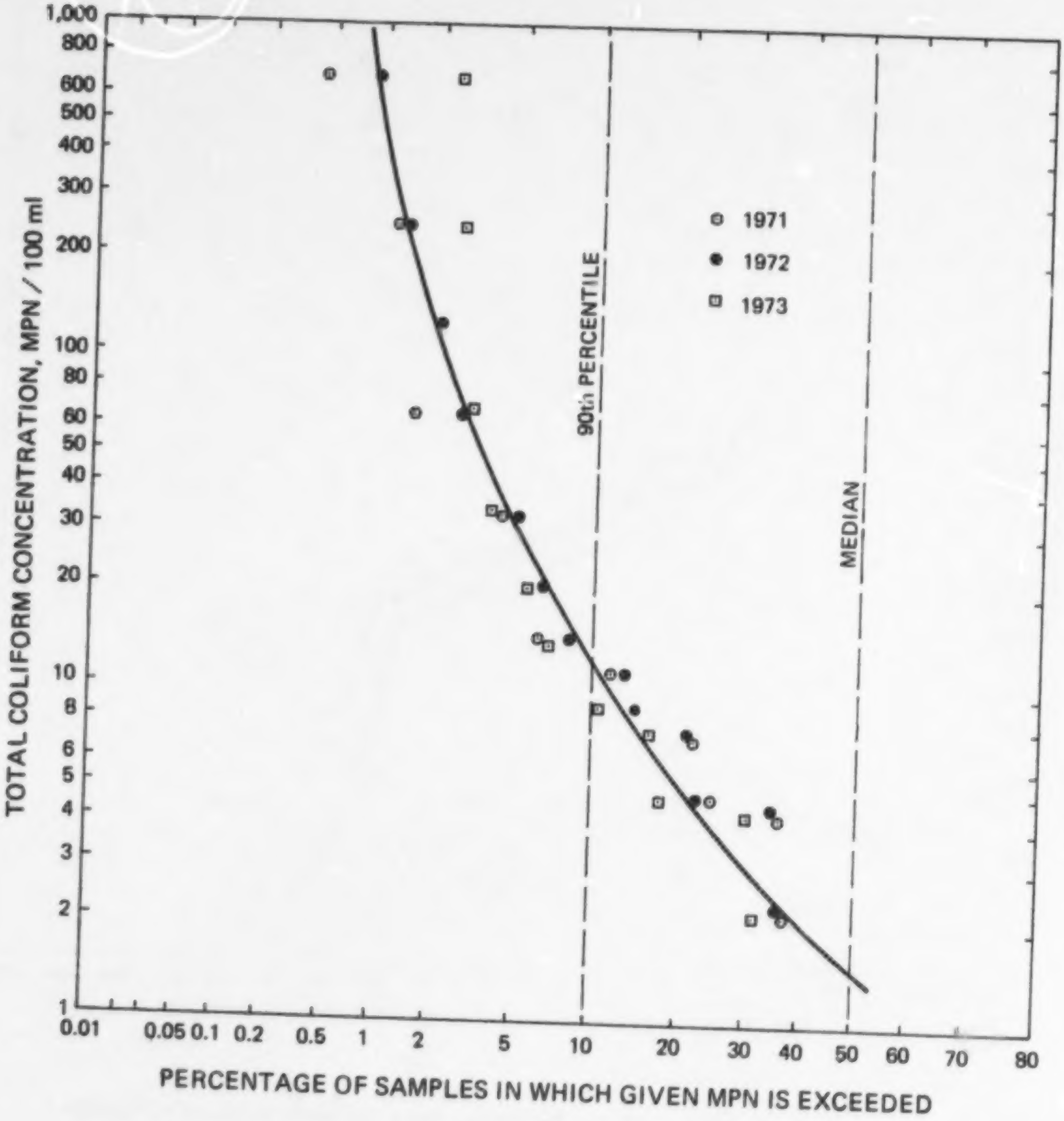

Figure 34. Probability curve for final effluent total coliform organisms.

the discussions on individual plant components, an overall evaluation of plant performance is presented, focusing on the relations among design loadings, experienced loadings, unit sizes, predicted plant performance, and actual performance.

Flows and Flow Metering

Influent flows to the Livermore plant are measured with two Parshall flumes, each with a throat width of 0.38 m (15 in.). These are located in the inlet works downstream from the comminutor and barminutor. Provision has been made in the influent structure to allow addition of a third flume in subsequent construction stages. Depth information from the meter is transformed to a flow rate and is recorded and totalized at the control panel in the operations building. All wastewater entering the plant must pass through the flumes. In addition, any flow subsequently diverted to the holding ponds from any point downstream of the inlet works must eventually be returned through the flumes and will thus be measured a second time. This error is insignificant on a long-term basis but may result in a discernible measured flow increase over a several-day period when the ponds are being emptied.

Wastewater flow leaving the plant is measured by a Cipolletti weir located downstream of the chlorine contact tank. The metered information is recorded and totalized at a nearby instrument panelboard. This flow meter has experienced erratic operation since its installation and has a long history of equipment malfunction and consequent repairs by factory personnel.

Monthly average, peak-day, and minimum-day raw wastewater flows (in mgd) recorded at the plant during the period of study are presented in Figure 35. The record shows an overall increase in flow from 1971 through 1972 with a subsequent decrease in 1973 and 1974. The precipitous decrease in early 1973 prompted a field check of the influent and effluent flow meters in July and August of that year. The results of those tests (disoussed in more detail below) indicated no reason for the sudden decrease; possible exfiltration from the sewer system remains the principal hypothesis.

It was noted in Section 5 that the design peak wet weather flow/average dry weather flow ratio was 0.79 m^3/sec ÷ 0.22 m^3/sec (18.0 mgd/5.0 mgd) = 3.6. In Figure 36,the probability curve for the ratio of daily peak flows to the average long-term flow is plotted for the period September 1973 through November 1974. This curve indicates that infiltration/inflow problems are much less severe than anticipated, which has been attributed to quality construction of new sewers and improved storm drainage systems. This has undoubtedly benefited plant operation, particularly with regard to secondary clarifier loadings, which will be discussed below. A typical diurnal flow curve for the plant influent is reproduced in Figure 37.

As noted above, the sudden flow decrease in early 1973 caused the City to undertake a program to evaluate the influent and effluent flow meters. The influent flow meter was checked two ways: (1) temporarily installing a rectangular weir and flow recorder upstream from the comminutor and barminutor and (2) operating the permanent recorder in conjunction with both the

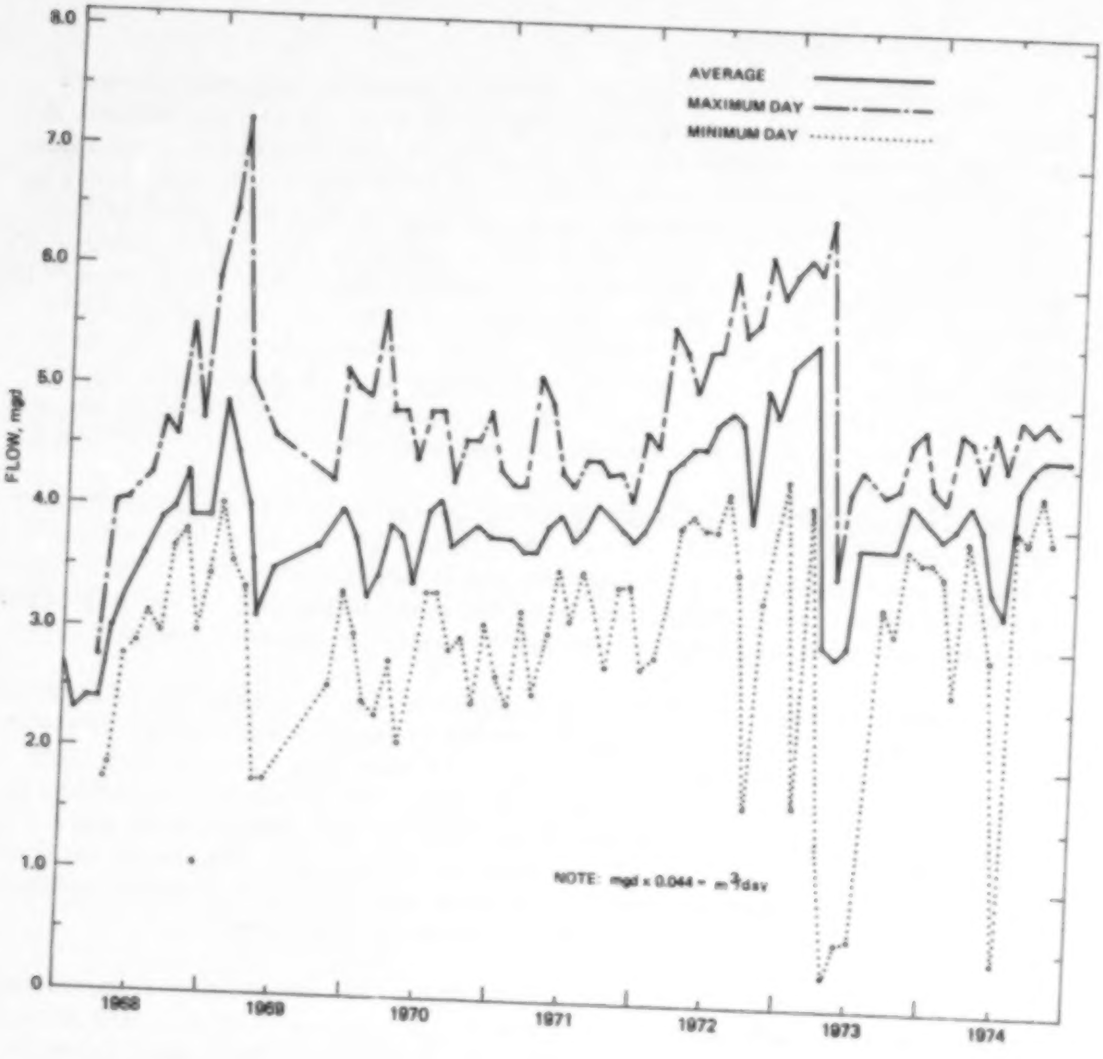

Figure 35. Monthly average, peak-day, and minimum-day flows.

permanently installed Parshall flumes and the temporary weir. The results of the study indicated that while the plant recorder was operating correctly, the Parshall flumes were incorrectly calibrated, resulting in a reading approximately 15 percent below the actual flow.

Flows recorded since September 1973 have been adjusted to account for the error, and all flow values prior to that date have been increased by 15 percent for use in this report. A dye tracer study undertaken in 1971 (and

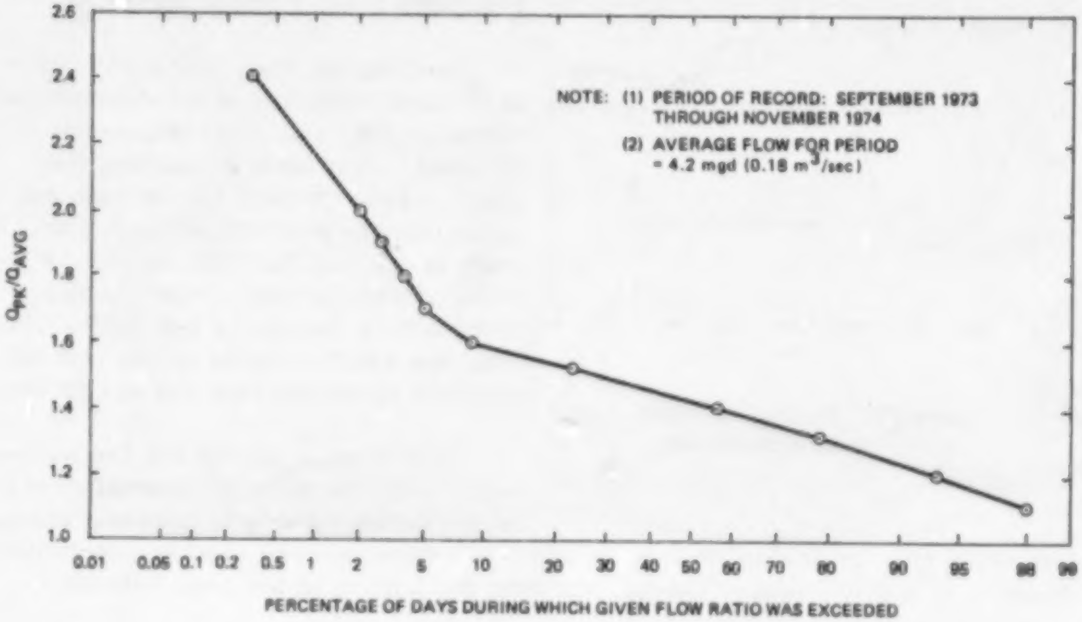

Figure 36. Probability curve for peak-to-average flow ratio.

discussed later in the subsection on disinfection) tends to confirm the 15-percent correction factor. Mean chlorine contact tank residence times measured in the tracer study were essentially equal to the calculated contact time adjusted for the 15-percent increase above the recorded flow.

The Cipolletti weir effluent meter was checked by installing a temporary recorder at the weir and comparing its results with those obtained through use of the permanent recorder. Over a 6-day period, daily flows measured utilizing the temporary recorder ranged from 11 to 53 percent greater than those indicated by the permanent recorder. It should be noted that effluent flows were low during this period (less than 0.066 m^3/sec or 1.5 mgd) because of diversions to the holding ponds, and that the greatest discrepancy, 53 percent, was associated with the lowest flow, 0.032 m^3/sec (0.72 mgd). Comparison of influent and effluent flows indicated that the Cipolletti weir was properly calibrated.

As a result of the metering studies, influent flow values were subsequently adjusted and plans were made to improve metering performance during the next plant modifications (undertaken in 1975).

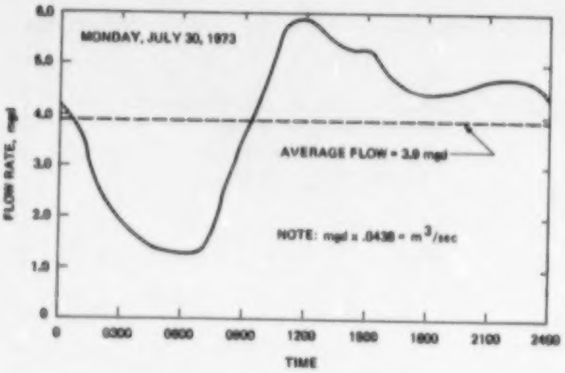

Figure 37. Typical diurnal flow variation.

Preliminary and Primary Treatment

Preliminary treatment at the Livermore plant consists of prechlorination, comminution, and preaeration-grit removal. Wastewater entering the plant passes through the comminutor or barminutor placed upstream from each of the two Parshall flumes. Prechlorination for odor control is practiced with a dosage of 2-8 mg/l. The chlorine addition point is the influent manhole upstream from the headworks.

Raw sewage pumps lift the wastewater into the influent channel ahead of the preaeration-grit removal tanks. Detention time at design flow is 36 min. A water-operated ejector, which discharges to a grit washer, removes grit from the hopper in the tank bottom.

Problems associated with the preaeration-grit removal tanks are occasional plugging of the grit ejector and formation of greaseballs which remain in the preaeration tank. Greaseballs must be removed by hand, which is unpleasant and at times hazardous. The average quantity of grit removed has ranged from 0.04 m^3/day (1.4 ft^3/day) in 1969 and 1970 to 0.15 m^3/day (5.3 ft^3/day) in 1974.

The primary sedimentation tanks (Figure 38) are located downstream from the preaeration-grit removal tanks in the same structure. These are conventional rectangular tanks 37.8 m (124 ft) long and 5.8 m (19 ft) wide with an average water depth of 2.7 m (9 ft).

Primary clarifier BOD_5 and suspended solids removals for 1968 and 1969 are plotted as a function of overflow rate in Figure 39. As indicated previously, the inclusion of waste mixed liquor in the sampled raw wastewater after 1969 makes calculated removals for those years erroneously high. The removals shown for 1968 and 1969 are somewhat lower than anticipated. Reasons for this are uncertain as the design overflow rate, detention time, and weir loading rate are within values which would be expected to provide better performance. A possible explanation is that the waste mixed liquor added at the headworks does, in fact, affect primary effluent quality.

Performance has not been of concern, partly because flows have been below the design value, but principally because influent BOD_5 and suspended solids concentrations have been far less than the design concentra-

Figure 38. Primary sedimentation tanks. These tanks performed more poorly than predicted, but low hydraulic and suspended solids loadings have minimized this effect.

tions of 300 mg/l, which corresponds to loadings of 5,670 kg/day (12,500 lb/day) at a flow of 0.22 m^3/day (5.0 mgd). Concentrations in 1968 and 1969 were in the 180-220 mg/l range, and from estimates of the effect of waste mixed liquor in more recent years, the influent values appear to have remained low. Thus, even though primary sedimentation removals have been less than predicted, loadings to the biological treatment units have not exceeded design levels. The reason for the difference between predicted and actual influent loadings is that the design concentrations were based on measurements using 8- or 16-hr composite samples, which resulted in higher measured concentrations. A comparison of predicted and actual removals for all the plant units is presented in the summary evaluation later in this section.

The principal operational problem associated with primary treatment has involved sludge removal using the air-lift pumps. Removal is controlled manually by the plant operator who adjusts the removal rate on the basis of visual observation of sludge entering the raw sludge sump. This has proved to be a poor method of control with underpumping or overpumping often occurring. Poor sludge removal control is an important problem at Livermore because of limited solids treatment capacity. A reduction in raw sludge solids content decreases the hydraulic and solids retention times in the mixed digesters,which in turn will reduce digestion efficiency. This aspect is dis-

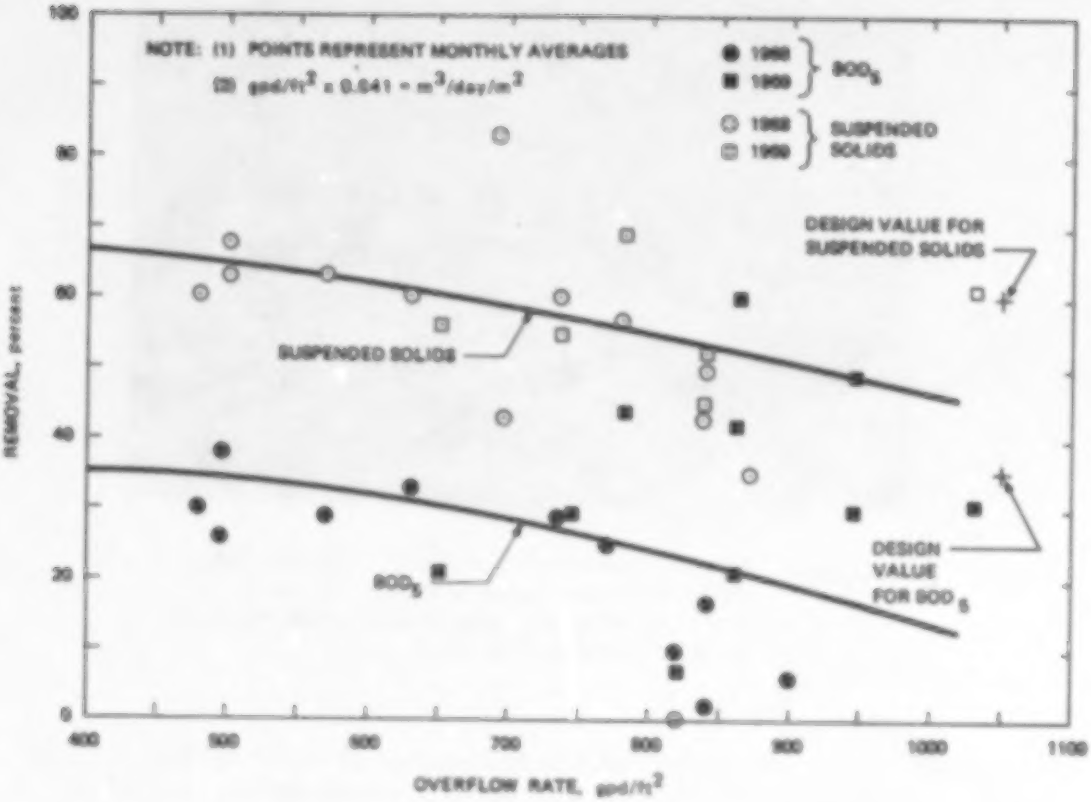

Figure 39. Primary clarifier performance.

cussed further in the subsection on solids handling and treatment. Improvement in sludge removal could be accomplished either by installing automatically controlled, positive displacement sludge pumps or by combining a more accurate determination of sludge solids content with the existing manual control.

Trickling Filters

The two rock-media trickling filters are used ahead of the activated sludge unit to enhance removal of organic material and allow nitrification to occur in the aeration tank. There are no clarifiers between the trickling filters and the aeration tank; thus, the measured BOD_5 loading to the activated sludge unit reflects sloughing of biological solids from the filter media surface. While this sloughed biological material will undergo endogenous respiration in the aeration tank and may thus affect air requirements, it does not constitute substrate for the activated sludge organisms in the usual sense.

Calculation of a food-to-microorganism ratio (F/M) should be modified to reflect this. Similarly, in calculating activated sludge solids retention times, the significance of including or excluding the effect of entering sloughed biological solids must be understood. These points will be discussed further in conjunction with the activated sludge unit and in Section 10, General Design Considerations.

Trickling filter performance for 1971 is summarized in Table 13. At a flow equal to about 80 percent of design, the average BOD_5 loading on the filters was 0.85 kg/m^3/day (53 lb/1,000 ft^3/day), or about one-half the design value. This again reflects the lower-than-expected raw wastewater BOD_5 concentrations experienced at Livermore.

TABLE 13. TRICKLING FILTER PERFORMANCE SUMMARY, 1971

Parameter	High month	Low month	Annual average
Flow, mgd[a]	4.1	3.7	3.9
Filter influent BOD_5 loading, lb/1,000 ft^3/day[b]	62	43	52
Filter influent concentrations, mg/l			
BOD_5	160	110	130
Suspended solids	98	60	79
Volatile suspended solids	88	33	61
Ammonia nitrogen	51	34	41
Filter effluent concentrations, mg/l[c]			
BOD_5	130	46	93
Suspended solids	140	74	110
Volatile suspended solids	117	63	92
Ammonia nitrogen	44	22	33
Reductions (increases), percent			
BOD_5	19	58	28
Suspended solids	(43)	(19)	(39)
Volatile suspended solids	(33)	(91)	(51)
Ammonia nitrogen	14	35	20

[a] mgd × 0.044 = m^3/sec

[b] lb/1,000 ft^3/day × 0.016 = kg/m^3/day

[c] Analyses made on unsettled effluent

Average BOD_5 removal across the filters for 1971 (based on unsettled effluent) was 28 percent. As shown in Appendix E, removals for other years in the operating period range from 15 percent in 1972 to 42 percent in 1969. Although a design removal of 50 percent was previously indicated in Table 3, there was little basis for this number, and it was in fact not relevant to the process design. It was noted in Section 5 that design of the activated sludge system was based on the assumption that equal filter and aeration tank volumes are equivalent in providing biological wastewater treatment.

Sloughing of biological solids from the filter rock is reflected in Table 13 by the increases in suspended solids (39 percent) and volatile suspended solids (51 percent). Some solids entering the trickling filter can be expected to be oxidized by the filter slime; thus, the numerical increase in solids concentration across the filter (110-79 = 31 mg/l in 1971) may be less than the actual solids yield of the filter biological process. Use of a trickling filter without a secondary clarifier has been so rare that little work has been done on the characteristics of the unsettled effluent.

Average ammonia nitrogen removal for 1971 was 8 mg/l, or 20 percent. This is largely due to incorporation of nitrogen into the biological cells produced. Approximately 1 kg of nitrogen is required for every 20 kg of BOD_5 removed by biological treatment.

Settling of Trickling Filter Effluent--

While precise analysis is difficult, it is useful to attempt to estimate the actual removal of BOD_5 across the filters along with the quantity of biological solids produced. To this end, Figure 40 was prepared which relates unsettled trickling filter effluent BOD_5 to the value obtained after allowing the sample to settle 1 hr (data available only for 1974). From the figure, the settled BOD_5 concentration is approximately one-half the unsettled value. A useful assumption is that the settleable material is sloughed biological solids. Relating Figure 40 with Table 13 indicates that the actual BOD_5 removal in 1971 was approximately $[130-(93/2)]/130 = 64$ percent rather than the 28 percent shown in Table 13. For comparison, as indicated previously in Table 2 (Section 4), an average BOD_5 removal of 66 percent was achieved across the trickling filter-secondary clarifier combination in 1964 at an average organic loading of 1.2 kg BOD_5/m^3/day (75 lb/1,000 ft^3/day).

Trickling filter BOD_5 removals (based on estimates of settled effluent concentrations) for the 7 yr of record are compared with trickling filter ammonia nitrogen removals in Table 14. The average estimated BOD_5 removal for the full period was 84 mg/l. Using the value referred to above of 1 kg nitrogen utilized per 20 kg BOD_5 removed, the expected ammonia nitrogen removal associated with this BOD_5 removal is 4.2 mg/l. This is very close to the measured average removal, 5 mg/l, implying that settling the trickling filter effluent for 1 hr provides a good estimate of actual BOD_5 removal.

Operation with One Trickling Filter--

Breakdown of the recently installed trickling filter distributor in 1968 and 1969 required that the plant be operated with only one filter for three separate

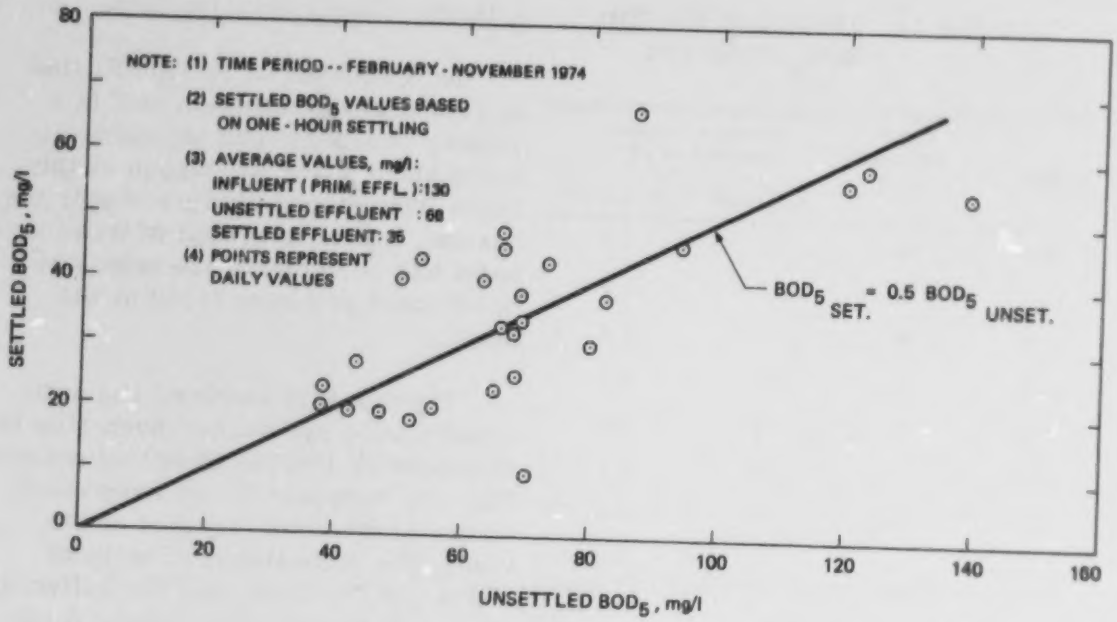

Figure 40. Effect of settling on trickling filter effluent BOD_5.

periods totaling 10 mo. During this time, the total flow was put through one filter, resulting in a substantially increased loading. The data summarized in Table 15 shows that while the average BOD_5 loading increased from 0.72 kg/m^3/day (45 lb/1,000 ft^3/day) with both filters in operation to 1.92 kg/m^3/day (120 lb/1,000 ft^3/day) with one filter out of service, BOD_5 removal decreased only slightly from 40 to 32 percent (based on unsettled effluent). The slightly higher suspended solids increase for one filter is probably due to initial sloughing under the higher hydraulic loadings.

Filter Circulation Sump--

It was noted in Section 5 that the use of fixed-speed pumps for delivering wastewater from the filter circulation sump to the activated sludge aeration tank is made possible by the use of an automatically controlled butterfly valve in a return line to modulate the flow. Difficulties have occurred with this approach; during peak and low flows, the control system tends to "hunt" and causes fluctuations in the flow to the aeration tank. Because of the inadequate air supply in the aeration tank, high loadings can adversely affect nitrification there. Continual checking by the plant operators is necessary to ensure that problems do not occur.

TABLE 14. TRICKLING FILTER BOD_5 REMOVAL

Year	BOD_5 removed, mg/l[a]	Ammonia nitrogen removed, mg/l	
		Measured	Estimated[b]
1968	93	-	4.7
1969	92	-	4.6
1970	90	6	4.5
1971	84	8	4.2
1972	75	2	3.8
1973	71	6	3.6
1974	86	5	4.3
Average	84	5	4.2

[a] $BOD_{5_{in}} - 0.5 \times$ unsettled $BOD_{5_{out}}$

[b] Based on 0.05 lb $NH_4^+ - N$ removed/lb BOD_5 removed

Activated Sludge Aeration Tank

As discussed in Section 5, the activated sludge aeration unit is a single, two-pass tank utilizing diffused air aeration for oxygen supply. Three fixed-speed blowers supply the system, and underdesign of these units has been one of the principal operational problems faced at the plant.

Despite this problem, the activated sludge system has been able to consistently provide excellent results. This has been due to the lower-than-expected loadings received at the plant, the consistency of influent flows and loadings, and the buffering effects of the roughing filter. A summary of activated sludge system performance for 1971 is presented in Table 16. The average and maximum-month secondary effluent BOD_5 concentrations of 10 and 17 mg/l, respectively, attest to the quality and consistency of performance. Secondary effluent ammonia nitrogen concentrations ranged from monthly values of 0.46 to 6.7 mg/l and averaged 1.7 mg/l for the year. The high value occurred in March and resulted from shutdown of one of the three blowers for repairs. The low value for air supplied, 45 m^3/kg (720 ft^3/lb) BOD_5 + NOD (nitrogen oxygen demand)* removed, also occurred during March.

The overall average of 1.7 mg/l is somewhat higher than should be expected from nitrification in an activated sludge system; it may be due to bleed-through of ammonia at peak loading periods, which will be discussed further below. Solids retention times varied from 4.2 to 8.6 days during the year. As will be discussed below, these values are retention times for the nitrifying organisms and ignore the effect of biological solids entering the aeration tank from the trickling filter.

---

*kg TKN x 4.56 = kg NOD. For this report, $NH_4^+ - N$ concentrations are substituted for TKN in calculating NOD since TKN concentrations were not routinely determined.

TABLE 15. TRICKLING FILTER PERFORMANCE--EFFECT OF REMOVING ONE FILTER FROM SERVICE

Parameter	Two filters[a]	One filter[b]
Flow, mgd[c]	3.3	4.1
$BOD_{5_{in}}$, mg/l	130	140
$BOD_{5_{out}}$, mg/l	78	95
BOD_5 loading, lb/1,000 ft^3/day[d]	45	120
Suspended solids increase, percent	21	29
BOD_5 reduction, percent	40	32

[a] Two filters operating: Jan-Sept '68; Jan, Feb, Aug-Oct '69

[b] One filter operating: Oct-Dec '68; Mar-July, Nov, Dec '69

[c] mgd x 0.044 = m^3/sec

[d] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

Air Supply--

Air for the activated sludge system at Livermore is supplied by three rotary-lobe, positive displacement blowers and air diffusion spargers. The blowers operate at a fixed speed and deliver 57 m^3/min (2,000 ft^3/min) each to the aeration tank and waste mixed liquor air-lift pumps. Originally designed to be operated with one blower as a standby unit, the high oxygen demand imposed by the nitrification process has dictated that all three blowers often be used simultaneously. The result has been that any lengthy shutdown for repairs causes a partial loss of nitrification. The most common maintenance problem has been bearing failure in the blowers. With replacement bearings kept on hand, repair usually requires 20 to 40 hr; longer shutdowns have occurred, however. In September 1974, for example, major repairs to one blower required 10 days to complete.

In addition to supplying the aeration tank, the three fixed-speed blowers also supply the air-lift pumps utilized for wasting mixed liquor from the system. Increasing or decreasing the number of operating blowers causes the

TABLE 16. ACTIVATED SLUDGE PERFORMANCE SUMMARY, 1971

Parameter	High month	Low month	Annual average
Plant influent flow, mgd[a]	4.1	3.7	3.9
Aeration tank influent concentrations, mg/l[b]			
BOD_5	130	46	93
Suspended solids	140	74	110
Ammonia nitrogen	44	22	33
Secondary effluent concentrations, mg/l			
BOD_5	17	5.2	10
Suspended solids	31	15	22
Ammonia nitrogen	6.7	0.46	1.7
Nitrate nitrogen	20	17	18
Reductions, percent			
BOD_5	93	85	89
Suspended solids	86	65	80
Ammonia nitrogen	99	84	95
Volumetric loading, lb/1,000 ft^3/day[c]			
BOD_5	28	10	21
BOD_5 +NOD	67	36	55
Food: microorganism ratio, lb BOD_5 applied /lb MLVSS/day[b,d]	0.27	0.12	0.21
Mixed liquor suspended solids, mg/l	2,320	1,700	2,010
Mixed liquor wasted, mgd	0.226	0.083	0.159
Solids retention time, autotrophic nitrifiers, days[e]	8.6	4.2	5.6
Return activated sludge suspended solids, mg/l	8,800	6,650	7,890
Return activated sludge flow, percent of influent flow	38	30	34
Air supplied, ft^3/(lb BOD_5 + lb NOD removed)[f]	1,470	720	990

[a] mgd x 0.044 = m^3/sec

[b] Based on unsettled trickling filter effluent

[c] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

[d] Based on a VSS/SS ratio of 0.79 measured in 1973-74

[e] Calculated as combined mass VSS lost per day in waste sludge and secondary effluent divided by mass MLVSS in aeration tank

[f] ft^3/lb x 0.0624 = m^3/kg

waste mixed liquor flow rate to decrease or increase, meaning that the flow rate must be checked and adjusted each time a blower is turned on or off.

The immediacy of nitrogenous oxygen demand in nitrification systems requires the use of peak rather than average ammonia loadings for design purposes. A typical daily BOD_5 + NOD loading to the activated sludge aeration tank is shown graphically in Figure 41. Although the average BOD_5 + NOD loading for the day was 5,070 kg/day (11,200 lb/day), the peak measured rate was 9,450 kg/day (21,000 lb/day), almost 90 percent above the average. For design, either instantaneous peak rates or short-term averages such as the maximum 3-hr average should be used.

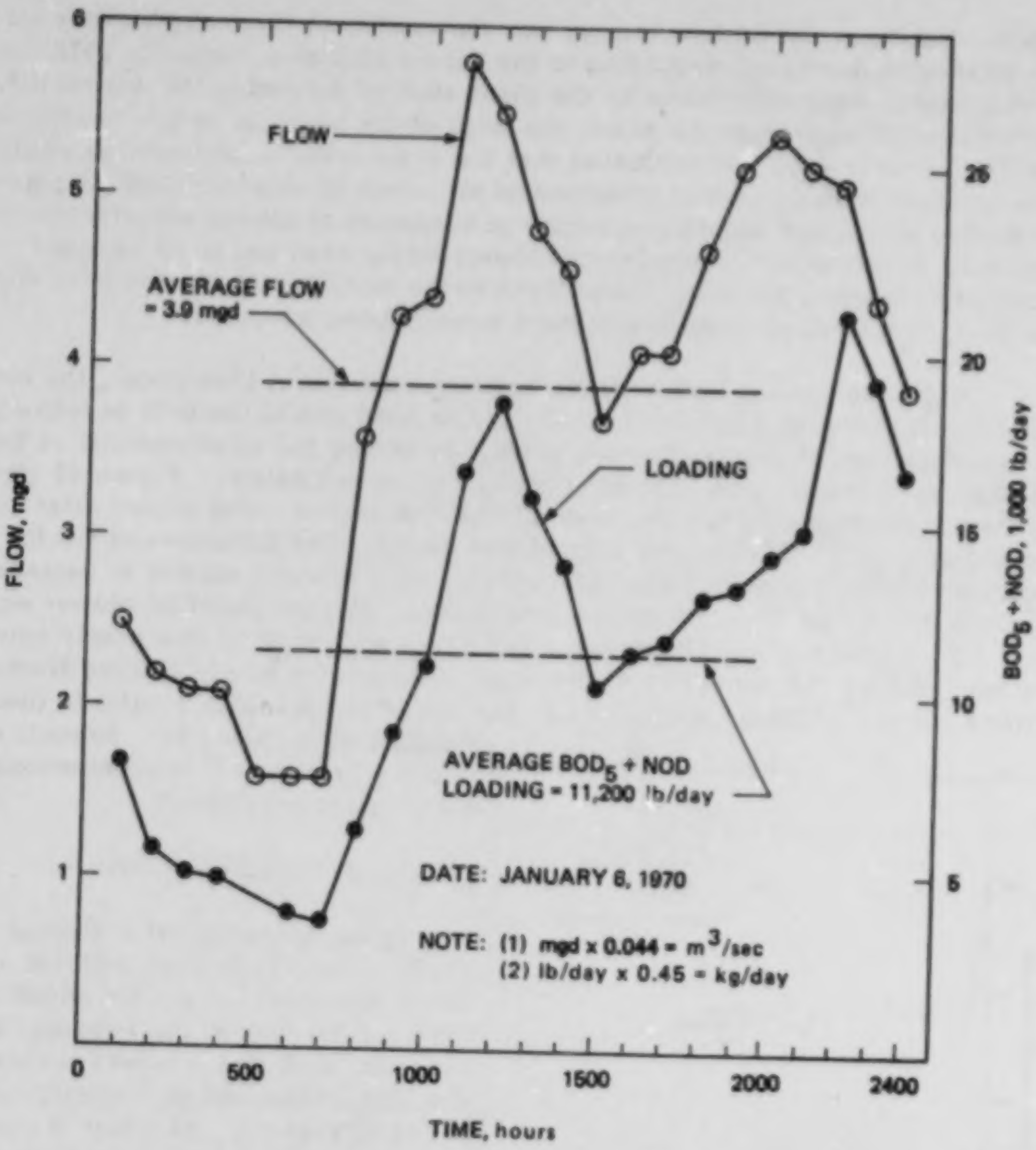

Figure 41. Flow and loading to aeration tank.

DO levels in the aeration tank are an important parameter in operating a nitrification system and, of course, directly reflect the quantity of air supplied and the loading of oxygen demanding substances. DO measurements at Livermore were initially taken near the effluent end of the two-pass aeration tank. Readings were taken hourly, and the number of operating blowers was increased or decreased according to the values obtained. In early 1972, investigations were undertaken by the plant staff to determine the desirability of taking DO measurements nearer the head of the aeration tank. The results of the several-week test indicated that the same level of performance could be obtained with an overall reduction of air usage of approximately 10 percent. A further advantage was the reduction in frequency of blower adjustments required. Previously, the number of blowers being used had to be changed three to six times per day. Subsequent to the modification in operating procedure, two or three adjustments have usually been adequate.

With plug flow operation of the aeration tank (as at Livermore), the lowest and thus limiting DO level occurs at the head end of the tank because the oxygen demand is greatest at that point. By taking DO measurements at this point, more precise and efficient control can be maintained. Figure 42 shows DO concentrations for the two ends of the tank over a 1-day period after use of influent-end DO levels for control was begun. The influent-end DO increased during the night when the loadings were low and started to decrease around 7 a.m. as the loadings became higher. At noon, the third blower was put into operation, and the influent-end DO level remained at a nearly constant level for the remainder of the day. Although the hourly loading fluctuations are not available for that date, the pattern is probably similar to that manifested in Figure 41. Records of monthly average DO concentrations are given in Appendix E.

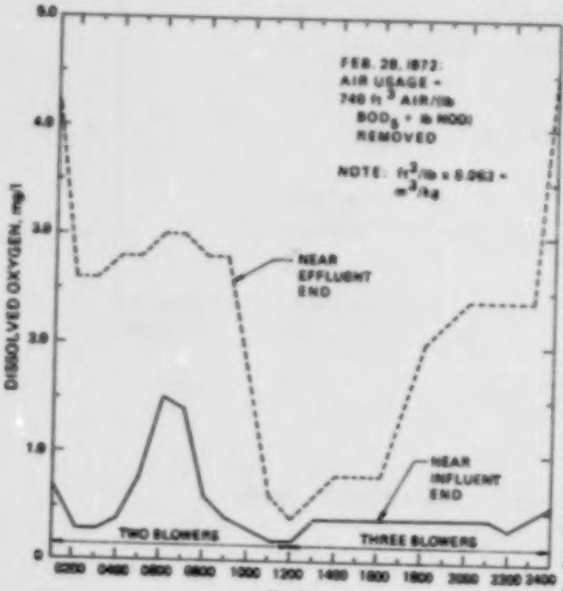

Figure 42. Aeration tank dissolved oxygen levels.

Residence Time Distribution--

In the conventional activated sludge process, primary effluent and return activated sludge are added to the aeration tank at the head end and leave the tank at the downstream end. The Livermore plant is normally operated in this manner, although it can be operated in the sludge reaeration or step aeration modes (see Section 5). To evaluate the effect of hydraulic load patterns, the plant was operated in the sludge reaeration mode for a short period in 1971. Return activated sludge was added to the head end of the aeration tank, and trickling filter effluent was added at the quarter point.

Nitrification was immediately adversely affected, and a return to the conventional mode was made after several days. It was concluded that the reduced contact time in the sludge reaeration mode of operation caused bleed-through of ammonia which was not oxidized during the shorter aeration period.

Measurements of MLSS levels during the period when sludge reaeration was utilized indicated little change in MLSS concentrations along the tank length. This indicates that considerable back-mixing occurs in the tank and suggests that under conventional operation the residence time distribution differs significantly from ideal plug flow, a condition in which all water particles are in the tank for approximately the same length of time. If back-mixing occurs, a significant fraction of the flow may leave the tank before ammonia oxidation has been completed. Ammonia bleed-through will be exaggerated by operation in the sludge reaeration mode because the aeration contact period will be significantly reduced.

Calculation of an "axial dispersion number" also indicates that the residence time distribution in the Livermore aeration tank is quite different from plug flow. This parameter is defined by D/uL, where D is the axial dispersion coefficient in m^2/hr, u is the mean displacement velocity along the tank length (including recycle) in m/hr, and L is the tank length in m. A valid empirical relation for D for both fine and coarse bubble diffused air systems is:[1]

$$D = 0.290\ W^2\ (A)^{0.346*} \qquad (1)$$

where: W = tank width, m, and

A = air flow per unit tank volume, standard $m^3/min/1,000\ m^3$.

The "axial dispersion number" is zero for true plug flow tanks and infinite for true complete-mix tanks. Tanks with $D/uL < 0.2$ are usually classified as plug flow reactors, while for complete-mix reactors, values are generally greater than 4.0.

For the Livermore plant, the "axial dispersion number" at $Q = 0.18\ m^3/sec$ (4.0 mgd) with 40 percent recycle is equal to 0.46. Because this is larger than 0.2, it indicates that plug flow is not well approximated in the aeration tanks.

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*$D = 3.122\ W^2\ (A)^{0.346}$, D in ft^2/hr, W in ft, A in standard $ft^3/min/1,000\ ft^3$.

Sludge Wasting--

The mixed liquor suspended solids (MLSS) level in an activated sludge aeration tank is controlled by wasting excess activated sludge from the system. At Livermore, wasting is accomplished from the mixed liquor line between the aeration tank and secondary clarifier (Figures 43 and 44). An air-lift pump delivers a portion of the mixed liquor to a constant-head tank consisting of a circular overflow weir and line (leading back to the mixed liquor line) with adjustable rings to control liquid head and a V-notch weir to measure and control waste activated sludge flow (refer to Figure 12 in Section 5). Waste mixed liquor is returned to the headworks, where essentially all the solids are removed during primary treatment so that there is little effect on subsequent units.

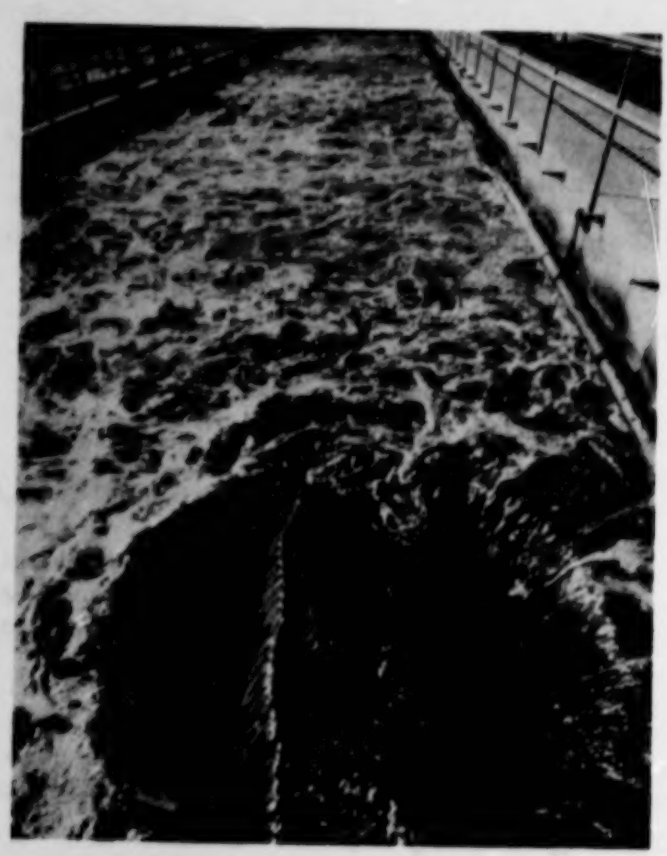


Figure 43. Mixed liquor leaving aeration tank over a circular weir.

In practice, flow over the constant-head tank circular weir is affected by flow from the air-lift pump, which in turn is dependent on the number of blowers operating. This has necessitated adjustment of the overflow weir each time a blower is started or stopped.

The more conventional method of wasting sludge is from the return activated sludge line. This can be done at Livermore, although metering is not available. Generally, wasting of return activated sludge has been confined to 1-hr periods, three or four times per month, in order to quickly reduce the aeration tank MLSS level when it has become too high.

Solids Retention Time--

A parameter often used in operating activated sludge systems is the solids retention time (often termed "mean cell residence time" or "sludge

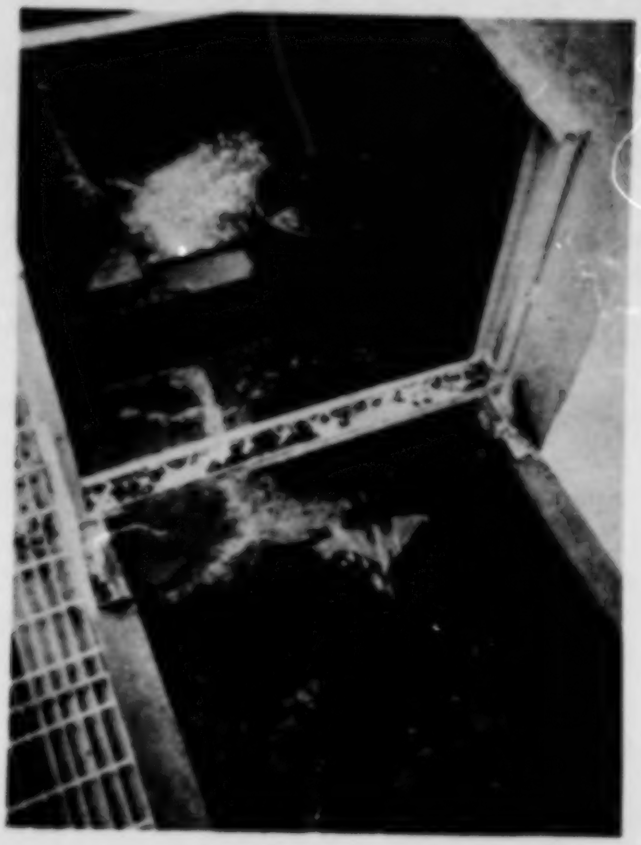


Figure 44. Constant-head tank in the mixed liquor line for wasting activated sludge.

age"), or θ_c. θ_c is defined as the inverse of the growth rate, $\mu - K_d$, of the organisms which oxidize the waste matter. In the conventional activated sludge process where oxidation of carbonaceous matter is the principal objective, the volatile suspended solids levels of the pertinent flow streams are usually taken as representing the heterotrophic organism concentrations of those streams.

The solids retention time can be computed for this situation by developing a mass balance around an activated sludge system as follows:

$$\begin{array}{lclclcl} \text{Rate of change} & & \text{Organisms} & & \text{Net organism} & & \text{Organisms} \\ \text{of organism} & = & \text{entering} & + & \text{growth rate} & - & \text{leaving} \\ \text{mass in system} & & \text{system} & & \text{in system} & & \text{system} \end{array}$$

$$Q \frac{dX_H}{dt} = QX_{H_1} + (\mu - K_d)VX_{H_{ML}} - (QX_{H_2} + Q_W X_{H_W}) \qquad (2)$$

where: Q = aeration tank influent flow rate, excluding sludge recycle,

$\frac{dX_H}{dt}$ = heterotrophic organism concentration rate of change in system,

X_{H_1} = heterotrophic organism concentration entering aeration tank in influent flow,

$X_{H_{ML}}$ = heterotrophic organism concentration in aeration tank,

X_{H_2} = heterotrophic organism concentration in secondary effluent,

X_{H_W} = heterotrophic organism concentration in waste sludge stream,

Q_W = sludge wasting flow rate,

μ = heterotrophic organism growth rate,

K_d = organism decay rate, and

V = aeration tank volume.

At steady state conditions, $\frac{dX_H}{dt} = 0$, and Equation 2 becomes:

$$\frac{1}{\theta_{c_H}} = \mu - K_d = \frac{Q}{VX_{H_{ML}}} X_{H_2} + \frac{Q_W}{VX_{H_{ML}}} X_{H_W} - \frac{Q}{VX_{H_{ML}}} X_{H_1} \qquad (3)$$

which further reduces to: $$\theta_{c_H} = \frac{VX_{H_{ML}}}{QX_{H_2} + Q_W X_{H_W} - QX_{H_1}} \qquad (4)$$

where: θ_{c_H} = heterotrophic organism solids retention time.

In the usual situation where primary effluent enters the system, X_{H_1} is taken as zero since it is assumed that very few organisms are entering the reactor. Where a roughing filter precedes the activated sludge unit, however, some significant fraction of the aeration tank influent volatile suspended solids are biological cells sloughed from the filter media, and X_{H_1} cannot be taken as zero.

The above derivation considers only aeration tank volatile suspended solids inventory in determining solids retention time. Some investigators prefer to calculate activated sludge solids retention time based on total secondary system volatile suspended solids inventory. In this case,the numerator of Equation 4 represents volatile suspended solids held in the secondary clarifier as well as in the aeration tank. Solids retention times calculated using the total system inventory will be larger than those calculated using aeration tank inventory only, substantially so if a deep sludge blanket is maintained in the secondary clarifier. The difficulty encountered in employing the total system inventory method is obtaining an accurate estimate of the average volatile suspended solids mass in the secondary clarifier over a 1-day or several-day period.

Because nitrifying organisms are not produced in a roughing filter application, the concentration of nitrifiers in the filter effluent is negligible, and the equation for the nitrifier solids retention time, θ_{c_N}, will be as follows:

$$\theta_{c_N} = \frac{VX_{N_{ML}}}{QX_{N_2} + Q_W X_{N_W}} \qquad (5)$$

where: $X_{N_{ML}}$ = nitrifying organism concentration in aeration tank,

X_{N_2} = nitrifying organism concentration in secondary effluent, and

X_{N_W} = nitrifying organism concentration in waste sludge stream.

Because the nitrifying organism concentrations can be taken as a constant fraction, f, of the respective heterotroph concentrations, Equation 5 becomes:

$$\theta_{c_N} = \frac{VfX_{H_{ML}}}{QfX_{H_2} + Q_W fX_{H_W}} = \frac{VX_{H_{ML}}}{QX_{H_2} + Q_W X_{H_W}} \qquad (6)$$

with the volatile suspended solids levels used for X.

Comparison of Equations 4 and 6 shows that the nitrifier solids retention time will be shorter than that for the heterotrophs. θ_{c_N} should be used when designing and operating coupled trickling filter-activated sludge systems for nitrification. Further discussions of this concept will be presented in Section 10.

As shown previously in Table 16, the monthly average solids retention times at Livermore in 1971 ranged from 4.2 to 8.6 days and averaged 5.6 days; these values were computed for the autotrophic nitrifiers (Equation 6) and are based upon use of the aeration tank volume as V.

The biological solids entering the activated sludge unit from the trickling filter cannot be ignored in process design. Although they do not directly affect the nitrifier solids retention time and although (as noted previously) settled BOD_5 values should be used in determining waste loadings to the activated sludge unit, the sloughed biological solids will affect aeration tank size. This will also be discussed further in Section 10.

Secondary Clarifier

The diameter of the secondary clarifier (Figure 45) at Livermore was limited by physical constraints at the site as noted in Section 5. Additionally, higher design overflow rates were generally used at that time. The result was a 27-m (90-ft) diameter clarifier with an ADWF overflow rate of 32.1 $m^3/day/m^2$ (787 gpd/ft^2). Average annual secondary effluent suspended solids concentrations are plotted against hydraulic overflow rate and solids loading rate, respectively, in Figures 46 and 47. The excellent correlations between these loading rates and clarifier performance suggest that clarifier size strongly affects performance at Livermore.

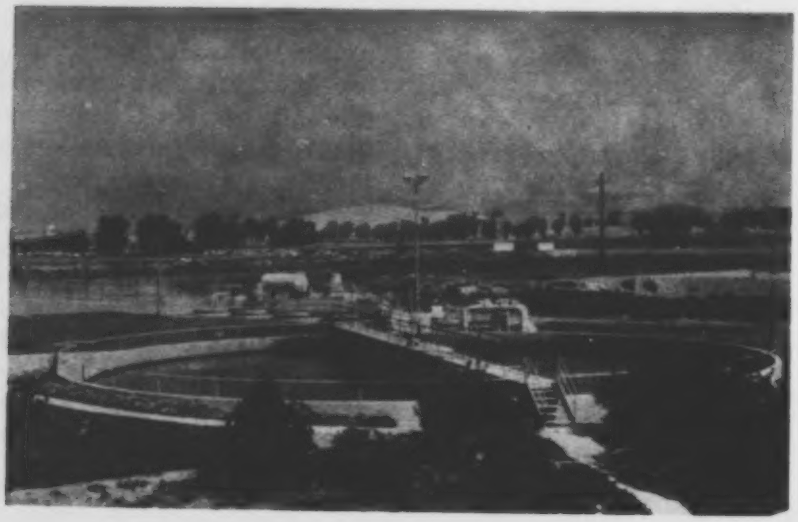


Figure 45. Secondary clarifier.

It should be pointed out that when the above parameters are plotted using daily or monthly averages, the scatter is significantly greater than found in Figures 46 and 47. Further, there is no strong correlation between secondary effluent BOD_5 concentrations and these loading rates or between BOD_5 and suspended solids levels. In fact, it has not been possible to correlate effluent BOD_5 values with any single loading parameter. This is probably due to the fairly narrow range of loadings experienced, the number of different parameters which can affect performance, and normal scatter. Nonetheless, the 5-yr average (1970-74) secondary effluent BOD_5 and suspended solids concentrations of 15 and 27 mg/l, respectively, are testimony to the excellent performance that the coupled trickling filter-activated sludge system is capable of providing.

An equipment breakdown in late 1974 necessitated shutdown of the secondary clarifier while repairs were made. During this period, trickling filter effluent was diverted to the holding ponds. The retaining bolts and brackets holding the sludge withdrawal lines in place over the circular trough on the

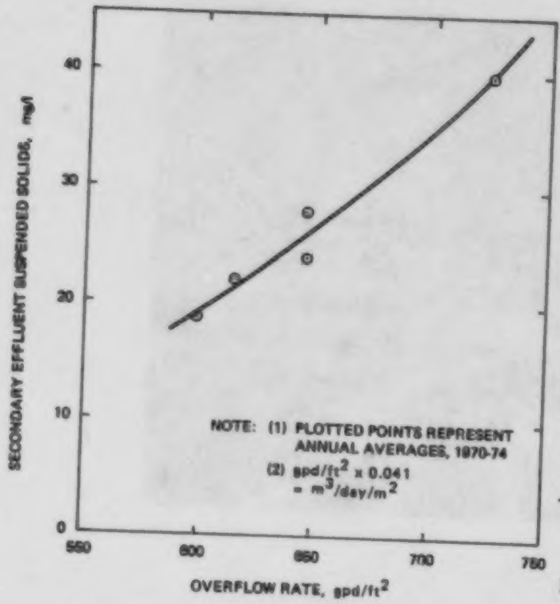


Figure 46. Secondary clarifier performance versus hydraulic loading.

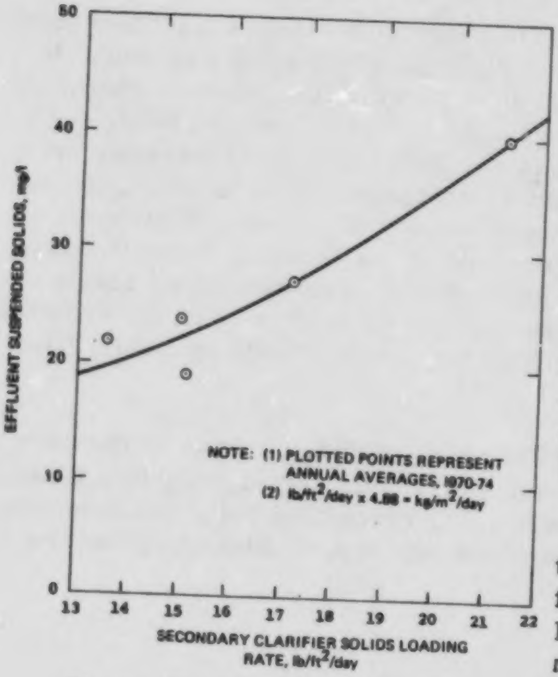


Figure 47. Secondary clarifier performance versus solids loading.

clarifier bottom had failed due to corrosion, allowing water from the higher levels of the tank to be withdrawn through the return activated sludge (RAS) lines. The problem was manifested by a low RAS solids content and high effluent suspended solids concentration. The cause of the problem was discovered when a portion of the mechanism extending above the waterline was noticed to be out of alignment. Clarifier dewatering and repair using stainless steel bolts and nuts followed.

Disinfection

Because the disinfection requirements for Livermore are unusually stringent, much effort has been directed over the years to improving the chlorination system used at the plant. Chlorine is added to nitrified secondary effluent as it enters the bypass manhole (see Figures 6 and 14 in Section 5) from the secondary clarifier. A 5.1-cm (2.0-in.) diameter pipe with 1.9 -cm (0.75-in.) holes spaced 7.6 cm (3.0 in.) apart for a length of 0.91 m (3.0 ft) acts as the chlorine diffuser. A two-paddle flash mixer placed in the manhole provides additional turbulence to aid the initial mixing step. The wastewater then flows through 76 m (250 ft) of 1.4-m (54-in.) pipe to the two-pass chlorine contact tank from which it is discharged to Arroyo Las Positas.

Originally, the plant was required to achieve a 5-day median total coliform concentration of 5.0 MPN/100 ml. In 1971, to develop consistent requirements for all dischargers in the area, the coliform requirement was modified (see Appendix C) by the Regional Board to 2.2 MPN/100 ml based on a 7-day median. Sampling frequency was changed from 5 days/wk to 7 days/wk, and as before, sampling was done at the period of peak flow, approximately 1:00 p.m.

Chlorination Control--

In the initial design, chlorination control was accomplished by utilizing a compound-loop system comprising a flow vacuum control unit and two amperometric chlorine residual analyzer-controllers (Figure 48). The flow vacuum control unit measured the wastewater flow rate upstream of the chlorine injection point to regulate chlorine dosage proportional to hydraulic flow. Immediately following chlorine injection, a wastewater sidestream was continuously pumped to the first residual analyzer-controller, which measured chlorine concentration and transmitted a signal to the chlorinator. This resulted in adjustment of chlorine dosage based on initial chlorine demand. At the end of the chlorine contact tank, another wastewater sidestream was continuously pumped to the second residual analyzer-controller (to measure chlorine residual prior to discharge) from which an override set-point signal was transmitted to the first residual analyzer-controller.

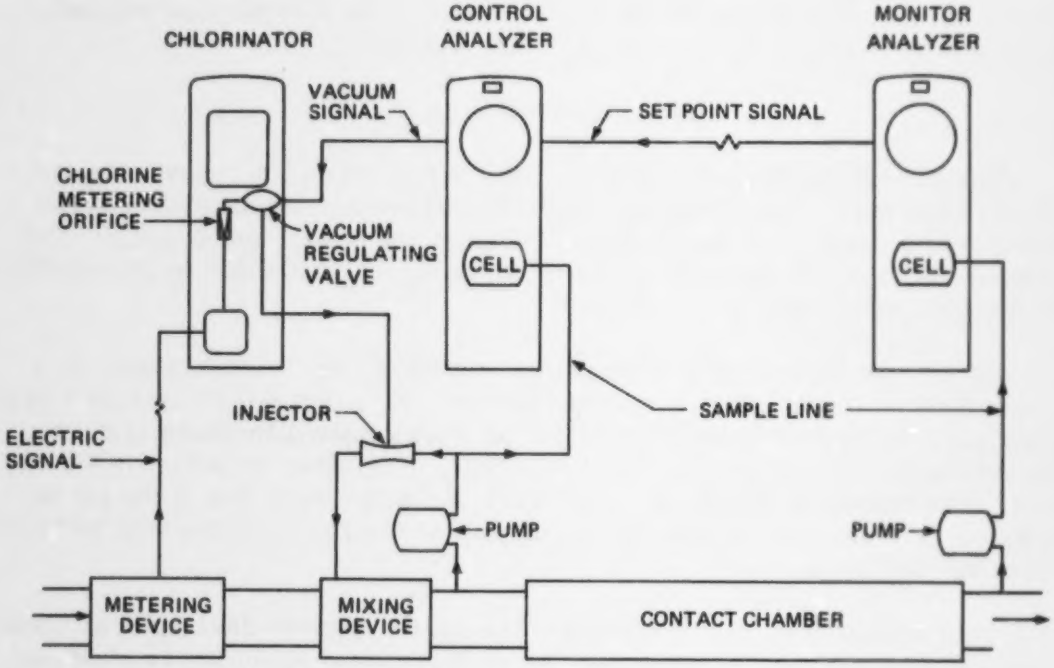

CHLORINATOR
CONTROL ANALYZER
MONITOR ANALYZER
VACUUM SIGNAL
SET POINT SIGNAL
CHLORINE METERING ORIFICE
VACUUM REGULATING VALVE
CELL
CELL
SAMPLE LINE
INJECTOR
ELECTRIC SIGNAL
PUMP
PUMP
METERING DEVICE
MIXING DEVICE
CONTACT CHAMBER

Figure 48. Original chlorination control system.

During the first full year of operation, 1968, many problems were encountered with this control system. Both sampling pumps required a complete overhaul, and the analyzers were a constant source of trouble, principally due to leaks and plugged tubing.

In the following year, an attempt was made by the plant staff to develop a new method of chlorination control based on the concept of using wastewater pH instead of chlorine residual as the control parameter. Addition of sufficient chlorine to water containing ammonia (approximately 10 parts by weight of chlorine is required for each part ammonia nitrogen) results in the production of hydrogen ions and a lowering of pH according to the following breakpoint reaction:

$$1.5\ Cl_2 + NH_4^+ \longrightarrow 0.5\ N_2 \uparrow + 4H^+ + 3\ Cl^- \qquad (7)$$

Chlorine added to wastewater after the breakpoint is reached reacts according to the following equation:

$$Cl_2 + H_2O \longrightarrow HOCl + H^+ + Cl^- \qquad (8)$$

This further lowers the wastewater pH. The product of this reaction, hypochlorous acid (HOCl), exists in equilibrium with its dissociation product, hypochlorite ion (OCl^-), according to the equation.

$$HOCl \rightleftharpoons OCl^- + H^+ \qquad (9)$$

The sum of the hypochlorous acid and hypochlorite ion existing in solution is known as "free available chlorine", which is a significantly better disinfecting agent than the combined chlorine compounds (principally monochloramine, NH_2Cl) that are the disinfecting agents occurring in non-nitrified wastewater effluents.

Of the two free available chlorine compounds, hypochlorous acid is a much stronger disinfectant than hypochlorite. Equation 9 indicates that hypochlorous acid is formed by a hydrogen ion concentration increase (pH decrease) which drives the reaction to the left. The effect of pH on HOCl and OCl^- distribution is shown in Figure 49.[2] It can be seen that if the pH is lowered to about 6.5, a significant fraction of the free chlorine will be in the hypochlorous form.

This principle forms the basis for the control system developed at Livermore in 1969 and used continuously since that time. Chlorine is added to the nitrified effluent at a rate which causes the pH to drop to about 6.5 in the chlorine contact tank. Although this control method could be automated, funds have not been available to purchase the required equipment and manual control has proven to be satisfactory.

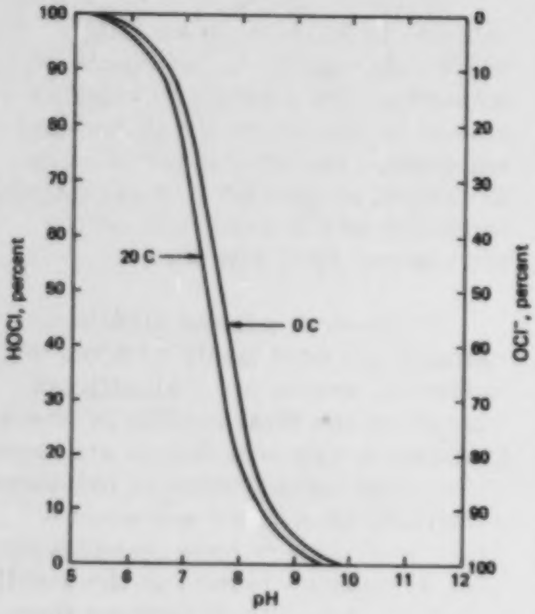


Figure 49. Relative amounts of HOCl and OCl at various pH levels (Ref. 2).

Disinfection performance obtained during initial experimental studies conducted with the pH control system is summarized in Figure 50. Total coliform survival ratio, y/y_o, is plotted against the product of chlorine residual, R and the chlorine contact time, T. Although values of y_o, the unchlorinated secondary effluent coliform MPN, were not measured during the study period, tests made at other times indicated values around 10^6 MPN/100 ml; this value was taken as y_o in preparing Figure 50.

Selleck and Collins[3] in laboratory studies of primary effluent chlorine disinfection, found the kill of coliform organisms followed the empirical relation:

$$\frac{y}{y_o} = A\,(RT)^{-3} \qquad (10)$$

where R is the total chlorine residual in mg/l and T is contact time in minutes. Selleck and Collins found that A = 64 for primary effluent, and this relation is shown in Figure 50 for comparison. Plant-scale disinfection studies at Rancho Cordova, California,[4] with non-nitrified secondary effluent yielded a similar relation with A = 4.0; this is also shown in Figure 50.

Although there are insufficient data available from Livermore to completely confirm the minus-three slope, this value has been assumed for the line drawn through the plotted points in Figure 50. The Livermore data indicate that for free chlorine disinfection of nitrified secondary effluent, A = 0.65.

Livermore has utilized pH control of chlorine addition since 1969 with excellent results, but difficulties can occur with this method. A principal problem stems from the fact that it is a very indirect control method and depends on a wastewater consistency, which does not always occur. Factors other than chlorine addition can cause a pH drop and an accompanying dosage decrease (although this can be compensated for by monitoring pH prior to chlorination).

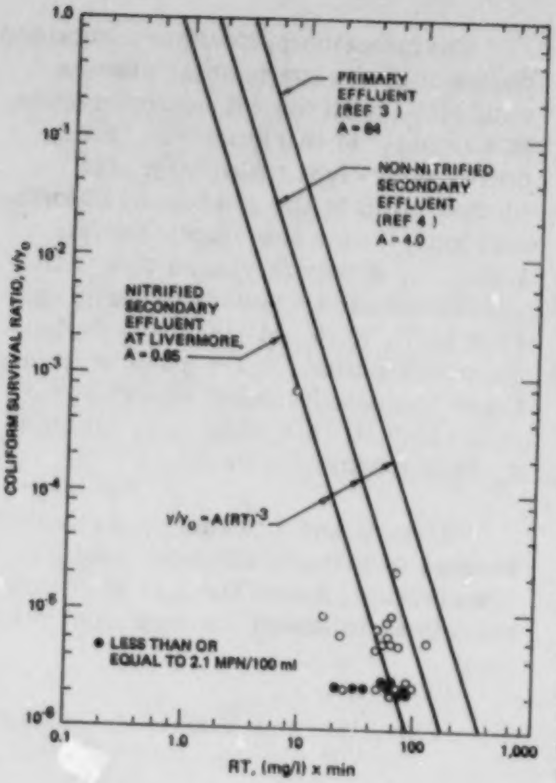


Figure 50. Disinfection performance.

Problems can also be caused by changes in wastewater alkalinity which, through its buffering effect, determines the quantity of chlorine needed to reduce the pH to a given set point. The pH-dosage relation developed at Livermore is essentially empirical and is dependent on the wastewater alkalinity there.

Wastewater pH and alkalinity changes are most likely to occur where industrial wastes are a significant fraction of the total loading or where infiltration/inflow problems are severe. The virtual nonexistence of industrial contributions and low wet weather flows at Livermore have undoubtedly been significant factors in the stability of the disinfection process there. Where wastewater characteristics change greatly with time, pH control of chlorination may be more difficult to operate successfully.

In addition to problems from changes in wastewater characteristics, it should be noted that reaching a particular pH value does not ensure that breakpoint chlorination has occurred. If high ammonia nitrogen concentrations are present due to a loss of nitrification for some reason, the set-point pH may be reached with the combined chlorine residuals alone. Cessation of nitrification at Livermore occurs primarily as a result of blower shutdown for repair. Thus, the problem becomes immediately apparent and action may be taken by the plant operators to prevent discharge of inadequately disinfected effluent (by an increase in chlorine dosage or diversion to the ponds). In other situations, however, signficant ammonia bleed-through may occur without immediately being apparent.

The data used in plotting Figure 50 are based on contact times and kills achieved in the 1.4-m (54-in.) diameter, 76-m (250-ft) pipe that precedes the chlorine contact tank. The test results, summarized in Table 17, indicate that at an average contact time of less than 10 min, a median coliform concentration of 2.1 MPN/100 ml was achieved. The 90th percentile value was

TABLE 17. SUMMARY OF pH CHLORINATION CONTROL STUDIES

Parameter	Value
Chlorine dose, mg/l	22.5
Chlorine residual, mg/l	6.8
Contact time, min	9.9
Secondary clarifier pH	7.4
Contact tank influent pH	6.6
Coliform concentration, MPN/100 ml	
Median	2.1
90th percentile	8.6

8.6 MPN/100 ml. These are very close to the mean and 90th percentile values shown previously in Figure 34, which is based on the full contact time provided by the pipe and the contact tank.

Chlorine Contact--

After chlorine is added at the bypass manhole, the wastewater flows through the 76-m (250-ft) long, 1.4-m (54-in.) diameter pipe and into the two-pass chlorine contact tank (Figure 51). Each pass is 30.5 m (100 ft) long, 3.2 m (10.5 ft) wide, with a water depth of about 4 m (13 ft). Mean detention time in the contact tank at design flow is 1 hr. The 1.4-m (54-in.) pipe adds another 8 min to the design contact time, and because flow in the pipe is very close to plug flow, the overall residence time distribution is very favorable for disinfection.

In the original design, plastic tubing was laid on the bottom of the contact tank to provide mixing through air addition. Mixing prevents settling of particulate matter and improves residence time distribution by helping to

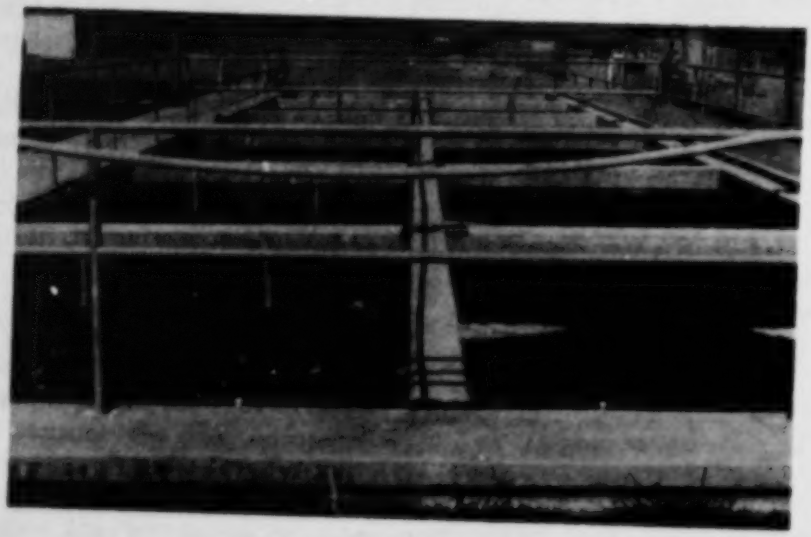


Figure 51. Chlorine contact tank. Air mixing takes place in pass on right.

eliminate dead space. Air addition also increases the DO concentration and raises pH. The high chlorine residual concentrations had an adverse effect on the tubing, and the air diffusion holes soon became plugged. Without air addition, the chlorine contact tank had to be cleaned at 1- to 2-mo intervals to remove settled matter; if it were not cleaned, disinfection performance was adversely affected.

In 1969, air diffusion spargers were added to the second pass to improve mixing. Surface spray heads were installed in the first pass to aid removal of floating material, principally filter flies, entering the contact tank. Periodic shutdown is still required for cleaning the first pass.

In 1971, a study was undertaken by the California State Department of Public Health to determine the residence time distribution of the chlorine contact unit, including both the 1.4-m (54-in.) pipe and the tank.[5] A slug of dye tracer was added at the bypass manhole where chlorine injection takes place. Fluorescence was then measured over time at the chlorine contact tank exit. The results are shown in Figure 52. Fluorescence, in arbitrary units, is plotted against t/T_{calc}, where t is time and T_{calc} is the calculated detention time, V/Q. Curves are shown for experiments with and without air added to the second pass. Each curve is based on two tests.

Performance is reflected by how closely the residence time distribution matches plug flow. In an ideal plug-flow system, all the tracer would leave the tank at $t/T_{calc} = 1.0$. Two useful measures of performance are the time required for tracer to be first detected, or t_i, and the time at which effluent tracer concentration is a maximum; the latter is termed the modal time, or t_m.

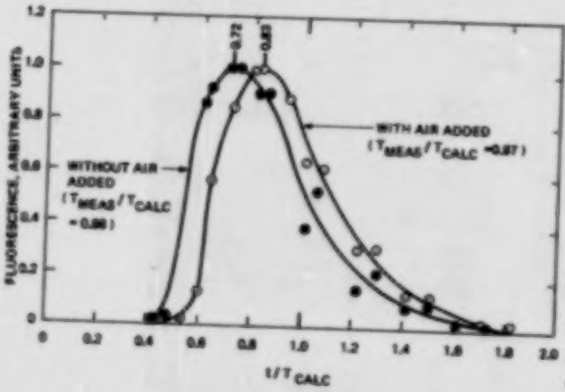


Figure 52. Residence time distribution for chlorine contact tank.

Figure 52 shows that dye first appeared in the contact tank effluent at $t/T_{calc} = 0.4$ to 0.5. Addition of air to the second pass improved the flow characteristics by increasing t_m/T_{calc} from 0.72 to 0.83.

The values for both t_i/T_{calc} and t_m/T_{calc} indicate excellent flow characteristics, far better than could be expected with the contact tank alone. The improvement is apparently due to the presence of the 1.4-m (54-in.) pipe ahead of the tank which provides flow characteristics very close to plug flow.

The mean residence time can be measured from the residence time distribution curve and theoretically should be equal to the calculated detention time, $T_{calc} = V/Q$. Because some tracer may remain in dead spaces of the tank for times much longer than t_m, measured mean residence times, T_{meas} are usually lower than calculated values. T_{meas}/T_{calc} values were 0.88 without air added and 0.97 with air; these indicate reasonable agreement between the flow measurement (corrected as discussed earlier) and dye tracer measurement studies.

The probability distribution for effluent total coliform MPN's was shown previously in Figure 34 and demonstrates the excellent performance of the Livermore Water Reclamation Plant in meeting the stringent disinfection requirements. Disinfection performance, including dosage, total chlorine residual, contact time, and median and 90th percentile coliform concentrations, is summarized in Table 18 for the 7-yr study period. Because samples for coliform determination are taken at the period of peak flow, the contact times shown represent the minimum for the day.

TABLE 18. DISINFECTION SUMMARY

Parameter	1968	1969	1970	1971	1972	1973	1974	Average
Flow, mgd[a]	3.7	4.0	3.8	3.9	4.6	4.1	4.1	4.0
Chlorine dose, mg/l	20	20	25	32	29	34	39	28
Chlorine residual, mg/l	>7.2[b]	>8.5[b]	>8.8[b]	14	16	17	18	16[c]
Contact time, min[d]	99	64	72	75	54	60	62	69
Coliform concentration, MPN/100 ml								
Median	2.1	2.1	2.1	2.1	2.1	2.1	<2.1	2.1[e]
90th percentile	32	15	19	15	15	15	-	15[e]

[a] mgd x 0.044 = m^3/sec

[b] Affected by maximum meter reading

[c] Based on 1971-74

[d] Based on maximum flow during day

[e] Median

Solids Handling and Treatment

In the plant expansion from a 0.11- to a 0.22-m^3/sec (2.5- to 5.0-mgd) flow capacity, no increase in anaerobic digester capacity was included. The two existing digesters were retained, and their operational mode was changed from the conventional primary-secondary operation to dual primary (or mixed) digesters. In order to treat the increased solids loading resulting from the increased plant flow and from incorporation of the activated sludge process in the plant flow diagram, two anaerobic sludge lagoons were constructed, each with a volume of 9,062 m^3 (320,000 ft^3). The original sludge drying beds were retained but have been rarely used because of long drying times and difficulties in cleaning the beds.

A summary of solids handling and treatment data for 1968-1974 is presented in Table 19. A more complete compilation is presented in Appendix E. The average hydraulic and solids retention time was 20 days. At a solids loading of 1.8 kg dry solids/m^3/day (0.11 lb/ft^3/day), approximately one-half the design value, volatile solids reduction in the digesters has averaged about 53 percent, slightly less than the assumed value of 60 percent shown previously in Table 3.

Solids handling ability has been a problem for the plant staff over the years, and odors from the sludge lagoons have often been noted. Because

TABLE 19. SOLIDS HANDLING AND TREATMENT SUMMARY

Parameter	1968	1969	1970	1971	1972	1973	1974	Overall average
Sewage flow, mgd[a]	3.7	4.0	3.8	3.9	4.6	4.1	4.1	4.1
Sludge flow to digesters, gpd[b]	17,100	18,000	18,800	20,600	22,300	22,830	21,400	20,100
Sludge flow to lagoons, gpd[b]	18,400	17,500	18,700	20,300	21,900	22,600	21,200	20,100
Solids retention time, days[c]	23	22	21	19	18	17	19	20
Raw sludge total solids, percent	3.8	3.8	3.5	3.5	3.7	3.3	3.3	3.6
Raw sludge volatile solids, percent of total	80	82	84	84	83	83	84	83
Digested sludge total solids, percent								
Digester No. 1	1.8	2.2	1.7	1.6	2.0	1.7	1.6	1.8
Digester No. 2	2.3	2.1	1.5	1.6	1.7	1.4	2.3	1.8
Digested sludge volatile solids, percent of total								
Digester No. 1	67	67	71	71	70	69	71	69
Digester No. 2	68	68	71	71	71	68	72	70
Volatile matter reduction, percent								
Digester No. 1	49	56	53	54	52	54	53	53
Digester No. 2	46	54	54	54	51	55	53	52
Gas produced, 1,000 ft^3/day[d]								
Digester No. 1	21.7	15.3	14.1	12.7	14.2	9.4	11.1	14.0
Digester No. 2	6.9	11.4	12.6	18.4	16.4	32.7	16.8	16.5

[a] mgd × 0.044 = m^3/sec

[b] gpd × 0.0038 = m^3/day

[c] Nominal value. Grit and scum accumulation reduced usable volume by about one-third by 1974.

[d] 1,000 ft^3/day × 28.3 = m^3/day

the solids retention time is equal to the hydraulic retention time for mixed digesters, a low solids concentration in the raw sludge will adversely affect performance. This situation has occurred occasionally due to the difficulty in determining the solids content of sludge being withdrawn from the primary sedimentation tanks.

As was noted in the discussion of primary clarifier performance, solids withdrawn from the clarifiers are discharged into an open sump with an invert approximately 3 m (10 ft) below a grated opening. This sump is checked by the plant operator on an hourly basis, and the sludge withdrawal is increased or decreased accordingly. Because of the difficulty in judging the solids content in this manner, sludge underpumping or overpumping often occurs. In particular, the beginning of each operator shift seems to be associated with the occurrence of such problems.

The system was part of the original trickling filter plant and seems to have operated well under those circumstances. Operating data from 1964 shows an average raw sludge solids concentration of 5.9 percent. With activated sludge added to the plant flow diagram, average yearly values dropped to a range of 3.3 to 3.8 percent (Table 19), slightly lower than the values of 4 to 5 percent often cited as typical for combined primary and settled activated sludge.[6,7]

Digester operation has generally been satisfactory over the years, although upsets have occasionally occurred. When upsets have occurred, the digester has been emptied and the digestion process started again.

Digester heating is provided by burning gas produced in the process, although provision has been made for use of natural gas when required. In 1970,the sludge circulation lines were insulated to improve digester heating.

Although Livermore's digesters are operated in a completely mixed mode, mixing has apparently been insufficient to prevent some stratification from occurring. In 1973,both digesters were checked and it was found that undigestible solids and grease had accumulated in each tank, reducing usable volumes by one-third and making calculated detention times erroneously high. One was cleaned in 1974, and the other is scheduled for cleaning soon.

Digester gas production is summarized in Figure 53. Monthly average gas production is plotted against volatile matter destroyed. The design value of 0.94 m^3/kg (15 ft^3/lb) is indicated as a straight line. Problems with gas flow meters have plagued the plant over the years, usually resulting in low readings. In developing Figure 53, readings taken during a period when problems were occurring were not used.

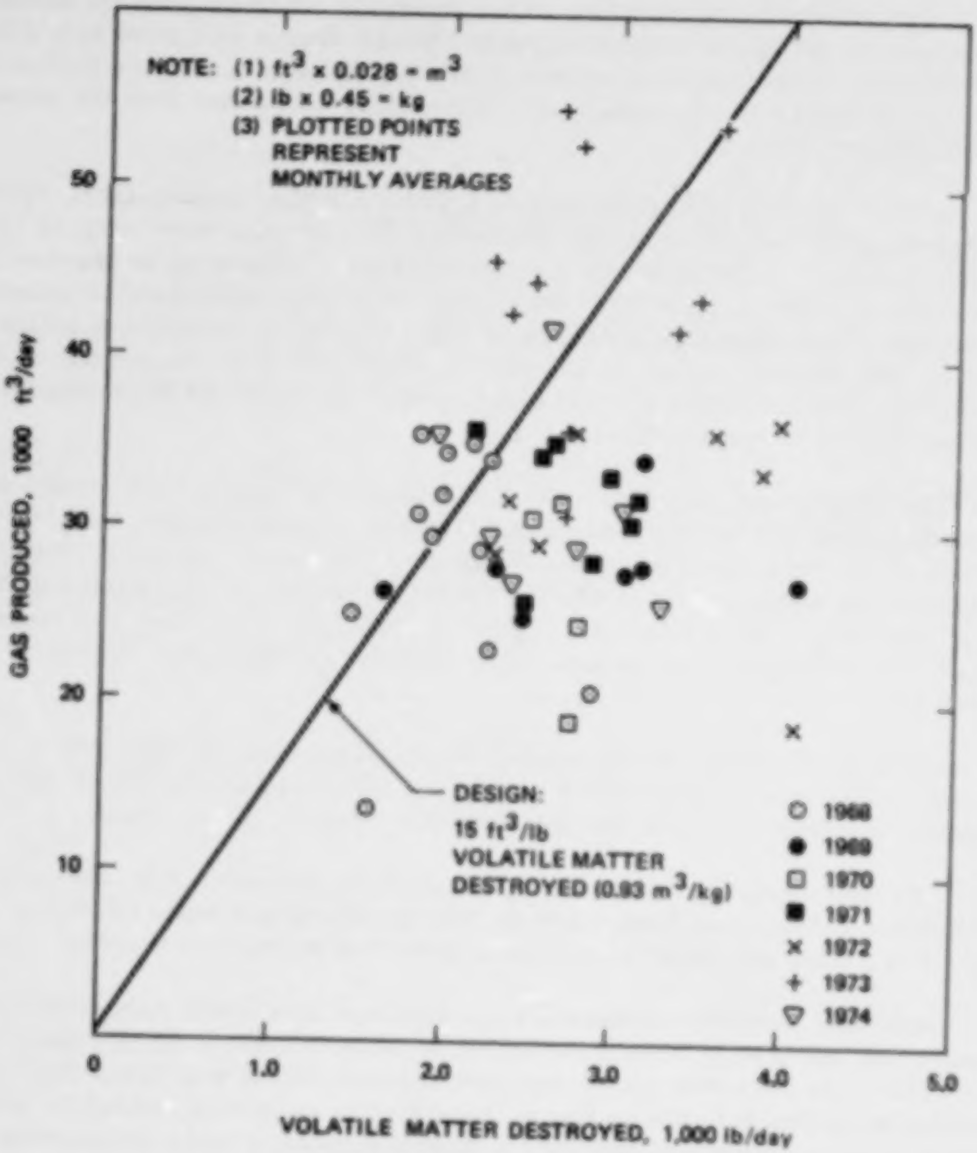


Figure 53. Digester gas production.

The sludge lagoons (Figure 54), which receive sludge from the two digesters, have a history of odor problems. Complaints are usually received on fairly calm summer nights, when the odors drift to residential areas to the east. On very windy days, which are common in the summertime, the wind apparently diffuses the odors and reduces the problem.

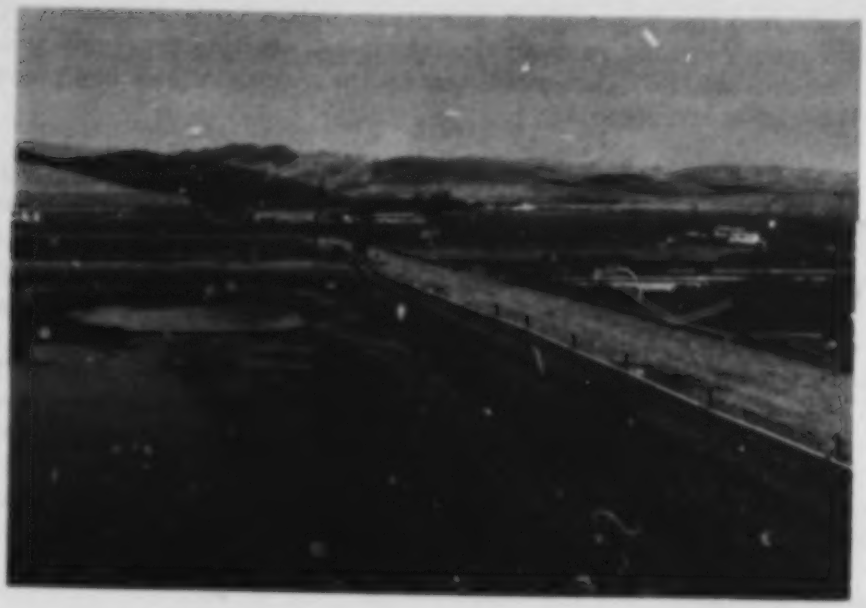


Figure 54. Sludge lagoons (left foreground). These lagoons have a history of odor problems. Piping for odor masking chemicals is located on bank. Little-used sludge drying beds are in right background.

To minimize the odor problem, the maximum water depth of 3 m (10 ft) is maintained in the lagoons. In addition, odor masking equipment was installed in 1969 adjacent to the downwind side of the ponds and consists of a diffuser system to spray deodorant chemicals into the air. At first, various masking agents were tried but without much success. At present, a patented deodorant chemical is being used which has effected an improvement, although odor complaints have not been entirely eliminated.

Studies with other sludge lagoons, principally at Auckland, New Zealand, indicate that to prevent odor nuisances, the lagoon volatile solids loading should be less than about 100 kg/1,000 m^2/day (20 lb/1,000 ft^2/day). In recent years, the loadings at Livermore have approached 300 kg/1,000 m^2/day (60 lb/1,000 ft^2/day).

Lagoon supernatant returned to the headworks is not regularly monitored for BOD_5 and suspended solids, but limited testing in 1969 indicated a BOD_5 level of 1,000 to 1,500 mg/l and a suspended solids concentration of 300 to

400 mg/l. During the first year of operation, fairly large amounts of lagoon supernatant were returned in short time periods, causing problems from slug loading. Since that time, however, the return procedure has been modified and problems have not recurred.

The four sludge drying beds were part of the original plant and were retained in the expansion to 0.22 m^3/sec (5.0 mgd). They are conventional sand drying beds with 0.6-m (2.0-ft) vertical concrete walls around the sides, 0.3 m (12 in.) of sand in the bottom, and perforated metal pipe designed to collect filtrate from gravel drains and return it to the headworks. Even during Stage 1 operations, some difficulties occurred because of inadequate capacity. The drains became plugged immediately, and evaporation and subsurface percolation were relied on to remove the water. After expansion,they have rarely been used because of the difficulties in cleaning the sand and because 3- to 4-mo drying times are required for 0.5 m (20 in.) of sludge. About 99 percent of the digested sludge has been treated in the lagoons. The dried sludge removed from the beds has been used as a soil conditioner by the City.

Reclamation Program

Reclamation of wastewater effluent for irrigation purposes has provided the City with an important supplemental water source in an area where importation has become necessary. Presently, about 162 ha (400 ac) are available to receive effluent, including 61 ha (150 ac) at the golf course, 20 ha (50 ac) at the municipal airport, and about 81 ha (200 ac) of privately owned farmland. Crops grown include sugar beets, cucumbers, squash, Sudan, and oats.

Wastewater effluent used for reclamation purposes in California must meet quality requirements set by the State Department of Public Health. For landscape irrigation of the golf course and airport, the applicable requirement states that the effluent must be "an adequately disinfected,oxidized wastewater". Adequate disinfection is assumed if the effluent total coliform MPN does not exceed 23/100 ml as a 7-day median.[8]

For surface irrigation of food crops which are to be processed in such a manner that pathogenic organisms are destroyed, a quality equivalent to primary effluent was required during the period covered by this report. This has recently been modified. Now, surface irrigation of any food crop requires the equivalent of secondary treatment and disinfection to a total coliform MPN of 2.2/100 ml as a 7-day median. The requirements now state that exceptions to the above requirements may be made where the food crop will undergo extensive commercial processing sufficient to destroy pathogenic agents.

The requirements for spray irrigation of food crops has been similarly modified. Previously, the requirement considered only food crops which would be processed to destroy pathogenic organisms and stipulated a disinfected, oxidized wastewater with a 7-day median coliform MPN not exceeding 23/100 ml. The requirement now applies to all food crops and stipulates that the wastewater be "adequately disinfected, oxidized, coagulated, clarified, (and) filtered". Disinfection requirements now stipulate that the 7-day median total coliform MPN not exceed 2.2/100 ml with a maximum of 23/100 ml allowed in no more than one sample per month. Exceptions may again be allowed for processed food crops.

Livermore Water Reclamation Plant effluent can meet any of the requirements except the coagulation and filtration requirements for spray irrigation of unprocessed food crops, and plant modifications recently completed will allow this requirement to be met.

The wastewater reclamation pattern shown for 1972 in Figure 55 is typical of irrigation demand in California. Little or no water is used in the wet months of October through March, but the demand is fairly constant from April through September.

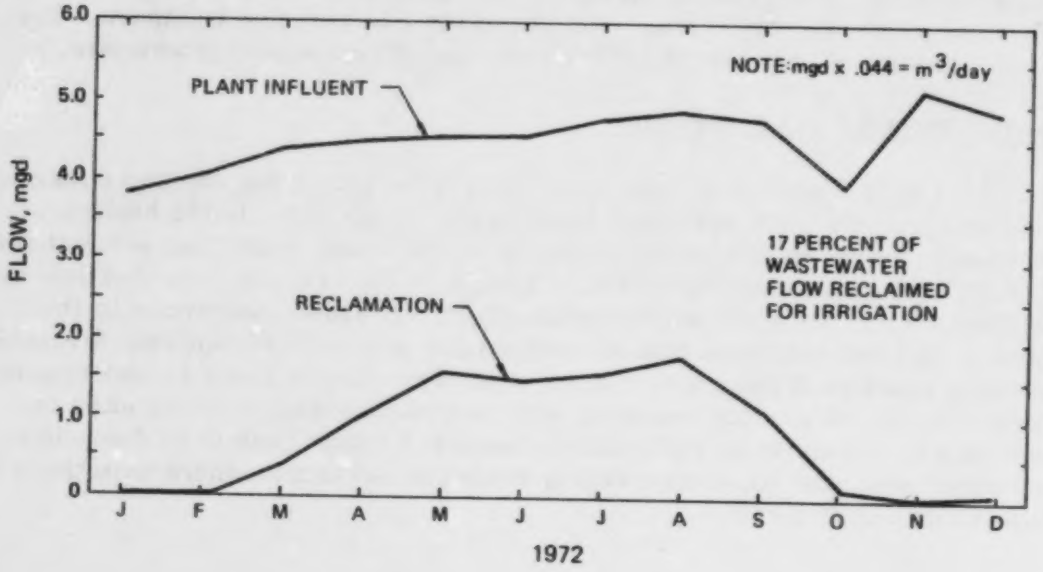


Figure 55. Typical annual reclamation pattern.

OPERATION AND PERFORMANCE EVALUATION SUMMARY

The information presented in this section indicates that, while some difficulties have been encountered in operating the upgraded Livermore Water Reclamation Plant, overall operation has been stable and has resulted in an effluent which is consistently low in contaminants.

Selected plant design parameters are listed in Table 20 along with corresponding performance data for 1968 and 1972, the years of lowest and highest measured influent flows, respectively. Many of the operational aspects discussed in this section are reflected in Table 20: low influent loadings, low primary treatment BOD_5 and suspended solids reductions, the influence of overflow rate on secondary clarifier performance, and the excellent final effluent quality. Because of the problem referred to previously of determining the significance of unsettled trickling filter effluent, loadings and removals for the combined biological treatment process are also shown in Table 20.

The discussion below summarizes major points made in this section with emphasis on problems which have been encountered. Items include (1) overall stability and performance, (2) influent loadings, (3) trickling filter effluent characteristics, (4) aeration capacity, (5) secondary clarifier performance, (7) disinfection, (8) bypassing, and (9) solids handling and treatment. The reader is referred to previous portions of this section where appropriate.

Overall Stability and Performance

More than 7 yr of operating experience have shown the coupled trickling filter-activated sludge system to be a highly stable and reliable biological treatment process configuration, capable of producing a nitrified effluent low in BOD_5 and suspended solids and amenable to disinfection to a very low total coliform MPN. The plant superintendent has expressed confidence in the process and has indicated that his experience with it has been very favorable. Summary plant performance data have been presented in Table 12 and Figures 32-34. Detailed monthly operating and performance data are presented in Appendix E. Operational difficulties discussed below have been due primarily to lack of adequate knowledge during design of the unique characteristics of this process configuration.

Influent Loadings

Reliance on previous 8- and 16-hr composite samples for determination of influent BOD_5 and suspended solids concentrations led to greatly overestimated projected BOD_5 and suspended solids loadings. While the design loading for both parameters was 5,670 kg/day (12,500 lb/day), annual average values have not exceeded about 3,400 kg/day (7,500 lb) excluding the effects of waste mixed liquor and lagoon supernatant in the influent.

TABLE 20. DESIGN AND PERFORMANCE

Parameter	Design	1968	1972
Influent			
Flow, mgd[a]	5.0	3.7	4.6
BOD_5, lb/day[b]	12,500	5,000	8,700[c]
Suspended solids, lb/day[b]	12,500	5,700	9,900[c]
Primary Treatment			
Overflow rate, gpd/ft^2 [d]	1,050	780	970
BOD_5 reduction, percent	35	22	43[c]
Suspended solids reduction, percent	60	55	58[c]
Trickling Filter - Activated Sludge Process			
Organic loading, lb $BOD_5/1,000\ ft^3$ trickling filter plus aeration tank volume/day[e]	35	19	21
BOD_5 reduction, percent	92	-[f]	88
Suspended solids reduction, percent	87	-[f]	75
Ammonia nitrogen reduction, percent	-	-[f]	97
Trickling Filters			
Organic loading, lb $BOD_5/1,000\ ft^3$/day[e]	100	50	62
BOD_5 reduction, percent	50[g]	33[g]	15[g]
Activated Sludge Aeration Tank			
Volumetric loading, lb $BOD_5/1,000\ ft^3$/day[e]	28[g]	20[g]	32[g]
Air supplied, ft^3/(lb BOD_5 + lb NOD removed)[h]	1,200[i]	-	821
Secondary Clarifier			
Overflow rate, gpd/ft^2 [d]	787	582	725
Effluent BOD_5, mg/l	15	-[f]	15
Effluent suspended solids, mg/l	15	-[f]	40
BOD_5 reduction, percent			
Based on activated sludge influent	86[g]	-[f]	86[g]
Based on raw wastewater	96	-[f]	93[c]
Suspended solids reduction, percent			
Based on activated sludge influent	86[g]	-[f]	67[g]
Based on raw wastewater	96	-[f]	90[c]
Final Effluent			
BOD_5, mg/l	20[j]	9.6	8.1
Suspended solids, mg/l	20[j]	9.5	22
Median total coliform organism concentration, MPN/100 ml	5.0[j]	2.1[k]	2.1[k]
Solids Handling			
Digester loading, lb dry solids/ft^3/day[l]	0.22	0.10	0.13
Volatile matter reduction, percent	60	48	52
Gas production, ft^3/lb volatile matter destroyed	15	14	10

[a] mgd x 0.044 = m^3/sec

[b] lb/day x 0.454 = kg/day

[c] Affected by waste mixed liquor and sludge lagoon supernatant recycled to headworks

[d] gpd/ft^2 x 0.041 = m^3/m^2/day

[e] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

[f] Secondary effluent quality not measured in 1968

[g] Based on unsettled trickling filter effluent

[h] ft^3/lb x 0.0624 = m^3/kg

[i] Based on BOD_5 only, two of three blowers operating

[j] Requirements set by California Regional Water Pollution Control Board, see Appendices B and C

[k] Monthly median

[l] lb/ft^3 x 16 = kg/m^3

These low loadings have greatly aided plant performance. In particular, removals in the primary sedimentation tanks have been slightly lower than anticipated, and the air supply for the activated sludge aeration tanks was significantly underdesigned. Serious operational problems would have developed if loadings had been as high as anticipated. In addition, the inadequacies in the solids handling and treatment portion of the plant would have been even more apparent.

Trickling Filter Effluent Characteristics

One of the most perplexing problems facing the designers of the upgraded plant was prediction of the relationship between the trickling filters and the activated sludge process. Very little information was available about the characteristics of the unsettled filter effluent that was to enter the aeration tank.

The design engineers avoided this problem by designing the biological treatment portion of the plant on the basis of total trickling filter plus aeration tank volume. The operating record of the plant has contributed to a greater understanding of the trickling filter-activated sludge interrelationship, but complete answers to some questions are still missing.

It is clear that much of the BOD_5 measured in unsettled trickling filter effluent is not waste material from the plant influent stream but rather biological solids sloughed from the filter media. Allowing filter effluent to settle 1 hr reduces the measured BOD_5 concentration by about 50 percent. Although this probably underestimates slightly the aeration tank influent BOD_5 concentration, it is a more accurate value than that based upon unsettled trickling filter effluent. The sloughed biological solids must be accounted for, however, in design of the activated sludge unit.

Aeration Capacity

The most serious operational problem has been a lack of adequate air supply for the activated sludge aeration tanks. The three fixed-speed blowers, which are capable of providing 57 m^3/min (2,000 ft^3/min) of air each, must all be used simultaneously during a portion of the day, eliminating the use of one blower for standby purposes. Blower shutdowns, usually resulting from bearing failures which require 20 to 40 hr to repair, cause a temporary loss of nitrification due to low DO concentrations. The resulting high effluent ammonia nitrogen levels make adequate disinfection more difficult to attain because of combined chlorine residual formation.

Because ammonia nitrogen is not sorbed by cell material to undergo oxidation later, it must be oxidized during the period that the wastewater is in the aeration tank. As the ammonia loading can vary greatly over a 24-hr period, the oxygen demand at peak periods can be quite high. It is possible that even with all three blowers operating at Livermore, ammonia bleed-through occurs at times. The bleed-through would not be apparent because measurements made on 24-hr composite samples would tend to mask this effect.

Secondary Clarifier Performance

Underdesign of the secondary clarifier has resulted in a secondary effluent suspended solids level somewhat higher than expected. The design ADWF overflow rate of 32.1 $m^3/day/m^2$ (787 gpd/ft^2) reflects the higher values often used at that time and the necessity of restricting the size to allow plant piping to be installed. The effects of hydraulic and solids loadings on effluent suspended solids levels were indicated preciously in Figures 46 and 47, respectively. Higher loadings have resulted in deterioration of effluent quality.

Disinfection

The restrictive, 7-day median, effluent coliform requirement of 2.2 MPN/100 ml (5.0 MPN/100 ml during the first portion of the study period) makes proper operation of the disinfection step extremely important. After experiencing much difficulty during the first 2 yr of operation using a conventional compound loop chlorination control system with amperometric residual chlorine analyzers, the plant staff developed a chlorination control method which involves adding chlorine until a specified pH, usually around 6.5, is reached. If the wastewater is low in ammonia nitrogen, free chlorine residuals will be formed and, at the low pH, hypochlorous acid (HOCl) will be predominant over hypochlorite ion (OCl^-). The former compound is a much stronger disinfecting agent, and the objective of this control method is the production of a high percentage of HOCl.

It was noted that there are inherent problems with this control method, specifically that a change in wastewater characteristics such as ammonia nitrogen concentration, pH, or alkalinity can affect the performance. The consistency of the raw wastewater quality at Livermore has allowed the plant staff to use this control method with singular success for more than 6 yr. However, caution should be used in considering this control method for other plants.

Chlorine contact is provided by a conventional chlorine contact tank with a 60-min detention time at design ADWF. Preceding the open tank is a 76-m (250-ft) long, 1.4-m (54-in.) diameter, full-flowing pipe from the secondary clarifier. Studies undertaken during initial experiments with pH chlorination control in 1969 indicated that adequate disinfection was being attained at the point where the 1.4-m (54-in.) pipe enters the contact tank. The advantage of additional contact time in the open tank is therefore uncertain, although it certainly provides an extra measure of protection in ensuring that the discharge requirements will be met.

Presently, air mixing is undertaken in the second pass of the contact tank to reduce dead space, raise the DO level, and raise the pH. No mixing occurs in the first tank with the result that contamination from solids deposition requires cleaning at 4- to 6-wk intervals to ensure that disinfection requirements can be met.

Bypassing

Conversion of portions of existing oxidation ponds to emergency holding basins during plant upgrading has greatly aided plant operation by allowing temporary discharge of raw or partially treated wastewater during periods of process unit shutdown. This is particularly valuable at Livermore because of the stringent discharge requirements and the absence of parallel units for the aeration tank and secondary clarifier. Wastewater stored in the basins is returned to the plant headworks during low flow periods after the plant is returned to service.

The principal problem occurring in bypassing is that primary or trickling filter effluent must pass through the chlorine contact tank to reach the holding basins. This contaminates the tank through deposition of solids, and it must be cleaned prior to returning it to service.

Solids Handling and Treatment

Plant expansion and the addition of the activated sludge process greatly increased solids production at the Livermore plant. Although sludge lagoons were added during expansion, sludge handling difficulties have occurred and have been manifested principally by odors from the lagoons.

The problem begins with the air-lift pumps which withdraw primary sludge and waste activated sludge from the primary sedimentation tanks. Installed originally to withdraw primary and trickling filter sludge from the clarifiers, the air lifts now pump a raw sludge with a solids content generally less than 4 percent. The two digesters, constructed with the original plant, are operated as parallel, mixed digesters whose effluent is discharged to the sludge lagoons. The low solids content and deposition of sand in a large fraction of the digester volume has reduced the solids retention time for the mixed digesters, resulting in a volatile solids reduction of around 50 percent in comparison with the design value of 60 percent. The lagoons receive nearly all this less than completely digested sludge as the sludge drying beds constructed with the original plant can accept less than 1 percent of the total. Continued digestion thus takes place in the lagoons, thereby precipitating periodic odor complaints from nearby residents.

Conclusions

The Livermore Water Reclamation Plant has provided excellent wastewater treatment through use of a coupled trickling filter-activated sludge biological treatment unit. The problems which have occurred are ones which can be eliminated in future plants of this type. Specific design suggestions for upgrading existing trickling filter plants to coupled trickling filter-activated sludge plants will be discussed in Section 10, General Design Considerations.

SECTION 9

TREATMENT COSTS

Presented in this section are summaries of capital and operating expenses for the upgraded Livermore Water Reclamation Plant. Capital costs cover detailed design, construction, and construction inspection; operating expenses include salaries, power and chemicals, operating supplies, direct maintenance, and administrative overhead. Costs are categorized where possible to aid evaluation.

CAPITAL COSTS

Because the upgrading of the Livermore plant involved both an expansion in flow capacity as well as modifications to the treatment flow daigram, it is useful to attempt to differentiate between these two categories in documenting capital costs. Although it is difficult to provide a complete breakdown of costs into the two categories of expansion and upgrading, this has been done to the extent possible.

Bids for construction were received on February 24, 1966, and the contract was awarded on February 28, 1966. The contract price was $1,067,000 (including $20,000 for contingencies) and corresponded to a San Francisco area ENR Construction Cost Index of 1150. Work was begun in March 1966 and completed in mid-1967. A breakdown of the bid was shown previously in Table 6 (Section 6). An installed cost breakdown for major equipment items was given in Table 5. The final total construction cost was $1,056,000 (rounded off) as summarized previously in Table 8.

Detailed engineering design work for the project totaled approximately $62,000, or 5.9 percent of the construction cost. This included periodic assistance during construction; full-time resident engineering services were not provided by the consultant. Also, preliminary planning costs are not included in the above figure. Costs borne by the City during construction for resident engineering and inspection were approximately $12,000, or 1.1 percent of the contract price. This includes only direct salary and fringe benefits for the single inspector employed by the City. Other costs, such as administrative overhead and direct expenses associated with inspection by the City, are not available. Adding these figures together yields a total project capital cost of $1,130,000.

Because the costs given above are for both upgrading and expansion, it is useful to present a previously developed estimate for expansion only, maintaining the original trickling filter-oxidation pond treatment scheme. These costs are given in Table 21 and are taken from a feasibility study prepared for the City by Brown and Caldwell in April 1964. Over half the total is for additional land for ponds.

OPERATION AND MAINTENANCE COSTS

Costs for operating the Livermore Water Reclamation Plant have been tabulated for the years 1968 through 1974 in Table 22. Salary costs include direct salaries plus fringe benefits for plant personnel. Power and chemicals comprise electric power, natural gas for augmenting digester heating, and chlorine. Operating expenses are normal supplies and direct expenses attributable to plant operation. Maintenance includes direct costs for repairs and contractual maintenance for specific items. Administrative overhead consists of the costs allocated to the administrative and clerical staffs of the City Department of Public Works.

TABLE 21. ESTIMATED COST FOR EXPANSION OF ORIGINAL LIVERMORE PLANT TO 5.0 MGD (0.22 M^3/SEC) WITHOUT UPGRADING

Plant Unit	Construction Cost, dollars[a]
Pretreatment and raw sewage pumping	12,100
Trickling filter	148,000
Secondary clarifier	182,000
Digested sludge lagoons	18,100
Chlorine contact tank	30,200
Oxidation ponds	113,000
Percolation beds for golf course irrigation water	12,100
Subtotal	515,500
Engineering and contingencies, 23 percent	119,600
Total	635,100
Purchase of 120 acres for oxidation ponds	725,000
Total project cost	1,360,100

[a] ENR Construction Cost Index = 1150

Total labor time at the Livermore plant is divided into five categories: operation, maintenance, repair, laboratory and monitoring, and yard work. A summary of the work load distribution for the 7-yr period from 1968 through 1974 is presented in Table 23. After 1969, the total number of man-hours worked increased very little, only 4 percent from 1969 through 1973. The final column shows the average percentage of the total allocated to each classification: 45 percent of the total

TABLE 22. OPERATION AND MAINTENANCE COSTS, LIVERMORE WATER RECLAMATION PLANT

Category	Annual cost, thousand dollars						
	1968	1969	1970	1971	1972	1973	1974
Salaries	101	126	134	150	155	160	178
Power and chemicals	30	33	39	43	48	63	84
Operating expenses	36	39	30	32	32	34	43
Maintenance and repair	12	12	14	15	15	17	36
Administrative overhead	17	22	34	25	27	33	40
Total	196	232	251	265	277	307	381
Percent increase	-	18	8	6	5	11	24

TABLE 23. WORK LOAD DISTRIBUTION

Item	1968	1969	1970	1971	1972	1973	1974	Average, percent of total
Time, man-hours								
Operation	7,210	7,805	8,605	8,820	9,062	9,150	8,714	45
Maintenance	2,506	2,160	2,923	2,567	2,748	2,997	3,795	15
Repair	2,450	2,855	2,800	2,630	2,695	2,963	3,714	15
Laboratory and monitoring	2,801	3,214	3,272	2,913	2,570	2,895	3,004	15
Yard	1,844	1,585	1,916	1,600	1,762	1,544	1,632	10
Part-time, unclassified[a]	-	1,182	-	-	-	-	-	-
Total	16,811	18,801	19,516	18,530	18,837	19,549	20,859	100
Annual increase, percent	-	12	4	-5	2	4	7	4[b]
Average salary cost, dollars per man-hour	6.01	6.70	6.87	8.09	8.23	8.18	8.53	-
Annual increase, percent	-	11	3	18	2	-1	4	6[b]

[a] Summertime student employees.

[b] Average annual increase.

man-hr under operation; 15 percent each under maintenance, repair, and laboratory and monitoring; and 10 percent under yard work.

Normal staffing at the Livermore plant consists of a superintendent, seven operators, a laboratory technician, and a maintenance mechanic. Yard work is provided through the Department of Public Works as a direct cost. Clerical work is provided as an overhead item. The plant is staffed on a 24-hr, 7-day basis.

Annual quantities of labor, chlorine, and electric power are plotted along with average annual flow for the years 1968 through 1974 in Figure 56. Labor required for plant operation has been fairly stable over the years, as is to be expected. Power costs have been related to flow, also as expected. Chlorine usage rose sharply, however, increasing from just over 100,000 kg/yr (220,000 lb/yr) in 1969 to almost 215,000 kg/yr (470,000 lb/yr) in 1972, a 215 percent increase in 4 yr. This was due principally to an increase in chlorine dosage necessitated by the changing of the disinfection requirement from 5.0 to 2.2 total coliform MPN/100 ml in late 1971 and to the changeover to pH control of chlorine addition.

TOTAL ANNUAL COST

Capital costs and operation and maintenance costs can be combined to give a total annual cost or total treatment cost for any particular treatment plant. Common methods of expressing such costs are dollars/yr or cents/m^3 (cents/1,000 gal) of wastewater treated.

To determine total annual cost, the capital costs must be amortized at a specific interest rate over a period of time. An interest rate and payment period often used for planning purposes are 7 percent and 20 yr, respectively. Based on these values, the capital cost of $1,130,000 is equivalent to $107,000/yr.

The operation and maintenance cost for 1968 was $196,000 at a flow of 0.16 m^3/sec (3.6 mgd). Adding this figure to the annual cost of capital yields a total annual cost of $303,000/yr.

It is useful to attempt to adjust this figure by expressing capital and operating costs on the same basis. The capital costs are associated with an ENR Construction Cost Index for the San Francisco area of 1150 in early 1966. By mid-1968, the Index had risen to 1250, resulting in an adjusted capital cost of $1,228,000. The equivalent annual cost at 7 percent and 20 yr is $115,000/yr. Combining the 1968 operation and maintenance cost with the amortized cost results in a total annual cost of $311,000/yr in 1968 dollars.

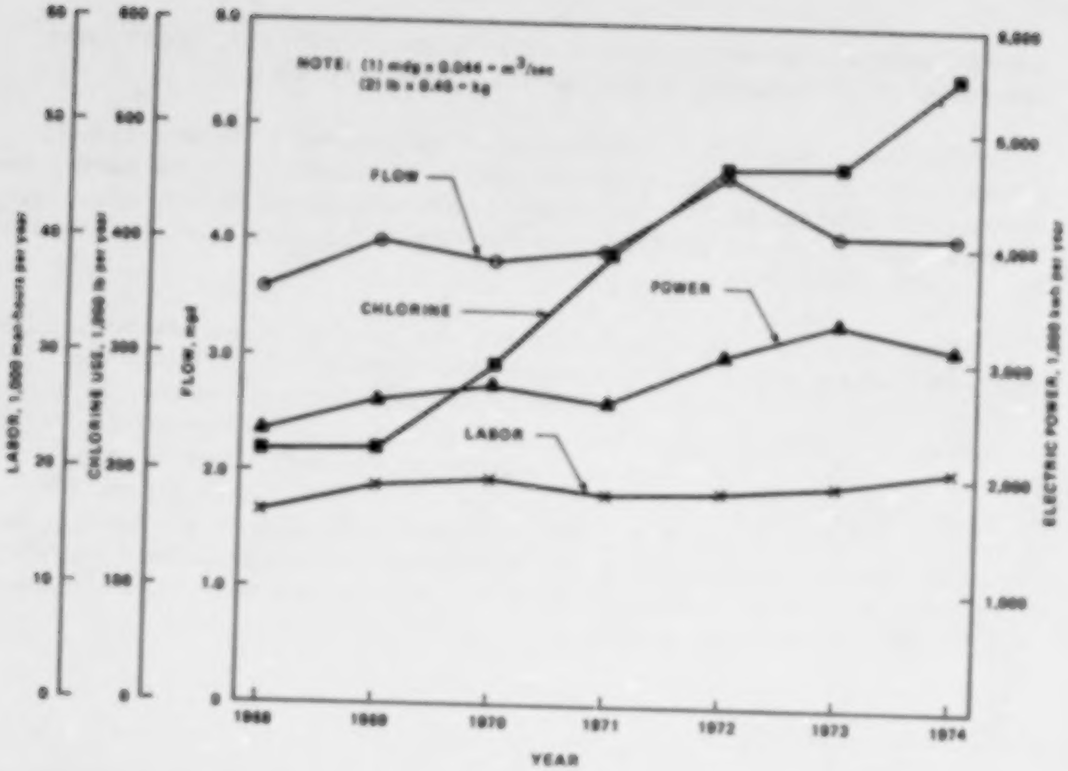


Figure 56. Flow and annual operating parameters.

Another often-used parameter for expressing total treatment cost is the cost in cents per unit volume of wastewater treated. In 1968, the average daily flow in the 0.22-m^3/sec (5.0-mgd) plant was 0.16 m^3/sec (3.6 mgd). Except for power and chemicals, most operating costs can be reasonably assumed to be independent of flow. Adjusting the 1968 operating cost to reflect the increase in power and chemical costs which would be expected at design flow yields an annual operation and maintenance cost of $208,000. The total annual cost in 1968 dollars is then $323,000/yr, or 4.7 cents/m^3 (17.7 cents/1,000 gal) based on design flow.

Although construction costs can be adjusted to reflect changes which occur over time by utilizing one of the several available construction cost indices, no similar simple adjustment can be made for operating costs. Thus, to estimate future total treatment costs, separate estimates must be made for construction and operation.

SECTION 10

GENERAL DESIGN CONSIDERATIONS

Experience gained from operation of the Livermore, California, Water Reclamation Plant can be of benefit in carrying out similar treatment plant modifications at other locations. Two coupled trickling filter-activated sludge plants are presently being constructed at Lompoc, California, and Corvallis, Oregon. Other operating plants include those at El Lago, Texas, and San Pablo, California. Except for San Pablo, these plants previously utilized conventional trickling filtration for secondary treatment and were upgraded after imposition of discharge requirements necessitating nitrification. The coupled trickling filter-activated sludge process is an alternative that should be considered by cities facing more stringent discharge requirements for their trickling filter plants.

The purpose of this section is to present design information on the combined trickling filter-activated sludge process developed from operation of the Livermore plant, design and operation of other plants, and information which has become available in recent years on design for nitrification in the activated sludge process. This is not intended to be a comprehensive design manual, but rather a brief outline of pertinent information with an indication of those areas where further data are needed. For a complete discussion of nitrification kinetics and process design, the reader is referred to the EPA Technology Transfer publication, Process Design Manual for Nitrogen Control.[9]

Presented in the following subsection are descriptions of the plants at Lompoc, Corvallis, El Lago, and San Pablo. Following these discussions is a brief review of nitrification process chemistry and biochemistry presented to provide a better understanding of the complexities encountered in design. Finally, the subsection on design considerations covers several aspects of plant design, including process design, aeration requirements, flexibility, reliability, use of existing facilities, and expected effluent quality.

ADDITIONAL TRICKLING FILTER-ACTIVATED SLUDGE PLANTS

Although this report deals primarily with the Livermore plant, data from other coupled trickling filter-activated sludge plants can be used to supplement the information developed at Livermore. The Corvallis and Lompoc upgraded plants were designed by Brown and Caldwell subsequent to the earlier Livermore upgrading. Certain changes in design parameters were adopted for these later plants. Other full-scale plants for which operating data are available include El Lago, Texas, and San Pablo, California. In addition to the coupled trickling filter-activated sludge process, the El Lago plant provides denitrification, phosphorus removal, and tertiary filtration.

Lompoc, California

The Lompoc Valley in Santa Barbara County, California, is the site of a coupled trickling filtration-activated sludge plant, presently under construction, which will provide consolidated wastewater treatment for four sewerage systems in the region. As at Livermore, the upgraded plant is being constructed on the site of an existing trickling filter plant and will utilize many of the available structures. Because of predicted flow increases, the anticipated loadings on the rock media filters would have been very high, possibly causing odors. Thus, the rock media filters are being converted to deep plastic media filters during the upgrading.

Lompoc is located approximately 16 km (10 mi) from the Pacific Ocean in a valley which is 19 km (12 mi) long and 5 km (3 mi) wide. In addition to the City of Lompoc, sewage treatment and disposal will be provided for the Federal Correctional Institution and two residential developments, Mission Hills and Vandenberg Village. In 1972, Brown and Caldwell undertook development of a regional wastewater management program for the Lompoc Valley. Because of long-term depletion of the local groundwater reservoir and deterioration of groundwater quality, a prime consideration in plan development was possible groundwater recharge using reclaimed wastewater. Recharge can be implemented either in special spreading basins or in the bed of the Santa Ynez River, which receives effluent from the treatment plant. Design effluent concentrations, whether intentional reclamation is practiced or not, are 15 mg/l BOD_5 and suspended solids, 0.1 mg/l ammonia nitrogen, and 2.2 MPN total coliform organisms/100 ml.

Previous Plant--

The Lompoc Water Pollution Control Plant provided secondary treatment by trickling filtration for a design flow of 0.077 m^3/sec (1.75 mgd), although measured flows had reached 0.092 m^3/sec (2.1 mgd) by 1972, resulting in noncompliance with discharge requirements. The plant was constructed in 1959 and replaced oxidation ponds which had previously provided treatment.

The plant was initially designed and constructed to operate as a two-stage trickling filter system with discharge to the Santa Ynez River. Between 1961 and 1966, however, a number of modifications to the plant were undertaken, including incorporation of the previously abandoned 43,000-m^3

(35 acre-ft) oxidation pond into the system to provide tertiary treatment, and rerouting of digester supernatant from the first-stage trickling filter to the primary sedimentation tank.

Upgraded Plant--

The Lompoc Regional Wastewater Reclamation Plant will provide treatment for 0.26 m^3/sec (6 mgd) ADWF and is planned for ultimate expansion to 0.53 m^3/sec (12 mgd). The flow diagram is shown in Figure 57. The plant will provide secondary treatment with nitrification and can be upgraded to include nutrient removal, filtration, and demineralization if future circumstances require such steps.

As at Livermore, emphasis has been placed on retaining existing facilities. The existing secondary clarifier will be converted to a sludge thickener, which will eliminate the need for increasing digestion capacity. Sludge lagoons will be placed on the site of previously existing drying beds, and the oxidation pond will be converted to an emergency holding basin, providing a detention time of about 2 days at design flow.

The total biological treatment volume is based on a unit loading of 0.53 kg BOD_5/m^3/day (33.2 lb/1,000 ft^3/day) of trickling filter plus aeration tank volume. Any two of the three blowers will provide 99 m^3 air/kg (1,580 ft^3/lb) BOD_5 + NOD entering the aeration tank (on a daily average basis). During the peak 3-hr loading period, the air supply rate with two blowers operating will drop to 82 m^3/kg (1,320 ft^3/lb).

The low bid for the Lompoc plant upgrading was $14.2 million, including $250,000 for contingencies. Construction was started in December 1974 and completion scheduled for mid-1977.

Corvallis, Oregon

The City of Corvallis is presently expanding and upgrading its municipal wastewater treatment plant from a 0.26-m^3/sec (5.9-mgd) trickling filter plant to a 0.42-m^3/sec (9.7-mgd) coupled trickling filter-activated sludge process with design effluent concentrations of 10 mg/l BOD_5 and suspended solids and 1.0 mg/l ammonia nitrogen. Provisions are being made for ultimate expansion to 0.76 m^3/sec (17.3 mgd), which is expected to serve the community beyond the year 2000.[10]

The principal objectives of the Corvallis plant and the problems found there are somewhat different from those at Livermore. Corvallis is situated on the Willamette River in western Oregon. For more than half a century, the Willamette has been the disposal site for liquid wastes from Corvallis and

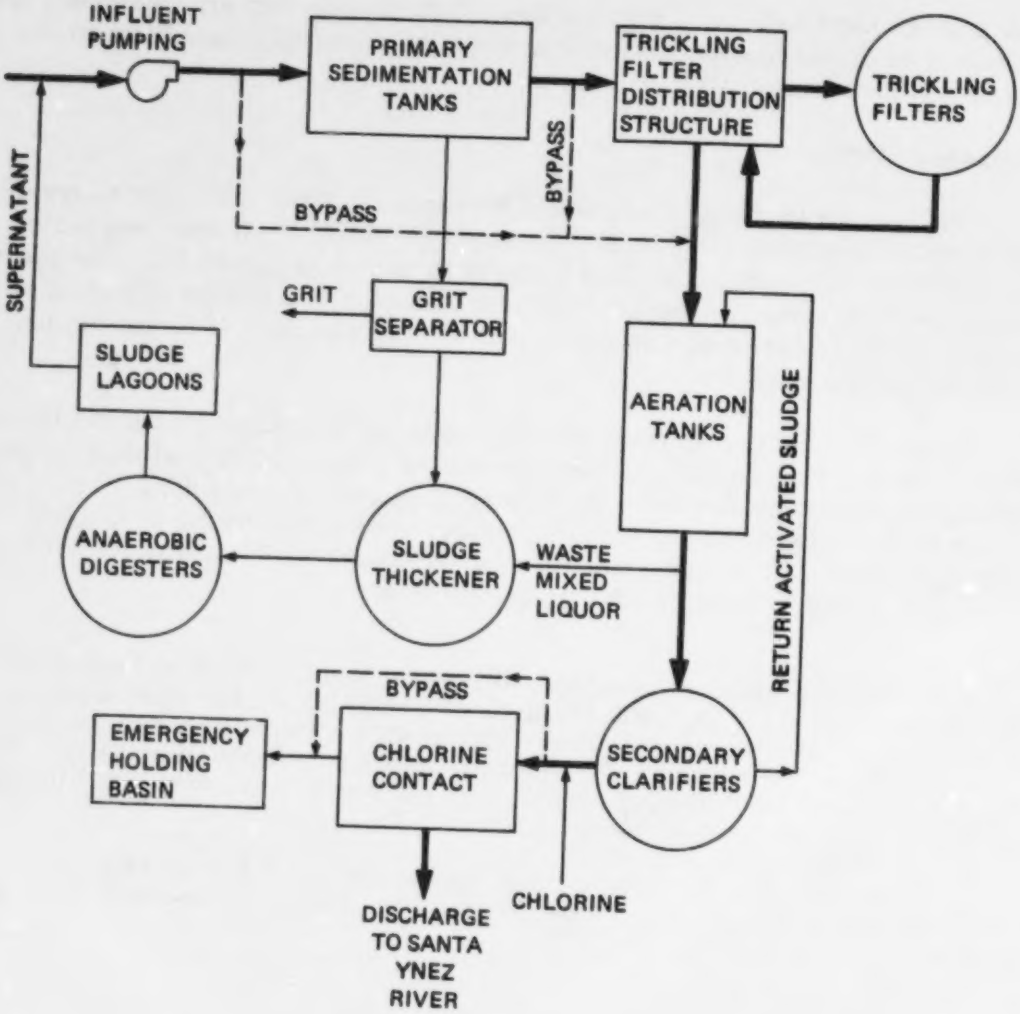

INFLUENT PUMPING
PRIMARY SEDIMENTATION TANKS
TRICKLING FILTER DISTRIBUTION STRUCTURE
TRICKLING FILTERS
BYPASS
BYPASS
SUPERNATANT
GRIT
GRIT SEPARATOR
SLUDGE LAGOONS
AERATION TANKS
RETURN ACTIVATED SLUDGE
ANAEROBIC DIGESTERS
SLUDGE THICKENER
WASTE MIXED LIQUOR
BYPASS
EMERGENCY HOLDING BASIN
CHLORINE CONTACT
SECONDARY CLARIFIERS
CHLORINE
DISCHARGE TO SANTA YNEZ RIVER

Figure 57. Lompoc, California, Regional Wastewater Reclamation Plant flow diagram.

other cities and industries along its route. As recently as 30 yr ago, substantially all wastes were discharged to the river without treatment. This practice resulted in gross pollution of certain stretches of the river, and during periods of low river flow, portions of the river were robbed of oxygen by the pollutants to a point where the river could no longer sustain fish and other desirable aquatic life.

Through diligent action by local, state, and federal agencies, the trend has been reversed in the last 20 yr and the quality of the river has improved to the point where it has once again taken its rightful place as a major aesthetic and recreational asset. To provide continued water quality improvement in the Willamette in the face of increasing wastewater discharges, the Oregon Department of Environmental Quality (DEQ) is planning to impose stiff limits on the oxygen-demanding constituents in wastewater effluents. Summer discharge requirements for Corvallis include mean BOD_5 and suspended solids concentrations of 10 mg/l. Although there is presently no requirement for ammonia removal, the plant is designed to achieve complete nitrification.

Previous Plant--

The first units of the existing sewage treatment plant were constructed in 1952 and consisted of an influent pumping station, an aerated grit chamber, a comminutor, a primary sedimentation tank, two floating-cover sludge digesters, and a control building. Primary effluent was discharged to the river after chlorination. In 1965, the plant was modified to provide secondary treatment by biological filtration. Principal components of the 1965 construction program included two trickling filters with a filter circulation pumping station, a secondary clarifier, and a third digester with a fixed cover.

During the peak of the food processing season in September, the BOD_5 load applied to the trickling filters has been up to three times the amount which the filters can treat effectively. As a result, the filters release odors which at times have reached serious nuisance proportions. Odor nuisance has not been a problem when the cannery is not operating.

Upgraded Plant--

In 1973, the City hired Brown and Caldwell to prepare a wastewater management program to allow the City to meet predicted future discharge requirements. The plan involved three principal elements: (1) construction of a stormwater overflow treatment and holding pond to reduce wet weather discharges from the City's combined sewers, (2) diversion of food processing wastes, which had periodically overloaded the municipal plant, to land treatment and disposal sites, and (3) conversion of the existing municipal plant to a coupled trickling filtration-activated sludge system for treatment of the community's municipal wastes.[10]

Experience obtained at the Livermore plant has allowed several important design improvements to be incorporated into the Corvallis facility. The design BOD_5 loading rate, in kg BOD_5/m^3 ($lb/1,000\ ft^3$) of biological treatment capacity (trickling filter plus aeration tank)/day, will be decreased from about 0.55 to 0.32 kg BOD_5/m^3/day (35 to 20 $lb/1,000\ ft^3$/day), a value which has been obtained in practice at Livermore. The design air supply will be increased to 86 m^3 air/kg (1,370 ft^3/lb) peak BOD_5 + NOD and sized to account not only for average nitrogenous oxygen demand, but for peak loading periods as well.

Secondary clarifier overflow rates will be reduced from those at Livermore to 21.2 m^3/day/m^2 (520 gpd/ft^2) at ADWF and will be based on a limiting wet weather rate of 61 m^3/day/m^2 (1,500 gpd/ft^2). To improve the settling characteristics of the mixed liquor, flocculation wells will be constructed at the center of each clarifier, providing 20-30 min of flocculation time at ADWF. Provision will be made for alum addition ahead of the clarifiers.

The existing secondary clarifier, which is unsuitable for settling activated sludge, will be used as a chlorine contact tank. Sludge removal equipment will be retained to periodically remove material escaping the secondary clarifier.

Solids handling and treatment will be unchanged at the new plant. One digester will be operated in the primary (or mixed) mode, and two will be operated as secondary digesters. Digested sludge will continue to be disposed of on land, and secondary digester supernatant will be returned to the plant headworks. Digester design loadings will be much lower than at Livermore, 1.8 kg dry solids/m^3/day (0.11 lb/ft^3/day) for the primary digester and 1.3 kg/m^3/day (0.08 lb/ft^3/day) for the secondary digesters, with an assumed volatile solids reduction of 55 percent.

The flow diagram for the upgraded Corvallis plant is shown in Figure 58. Instead of relying on holding basins for emergency situations, the Corvallis plant emphasizes multiple units, allowing a brief shutdown of one aeration tank, for example, with minimal effect on overall plant efficiency. Further reliability is provided by a separate facility for treatment of sewage overflows, which occur routinely during wet weather.

The low construction bid for the Corvallis plant was $7.7 million. Construction was started in November 1975 with completion scheduled in November 1977.

El Lago, Texas

El Lago is a small suburban community of 3,000 people located near the Lyndon B. Johnson Space Center in Texas. The agency responsible for wastewater treatment is the Harris County Water Control and Improvement District No. 50, which currently operates a 0.013-m^3/sec (0.3-mgd) treatment plant. In 1969, the District received an order from the Texas Water Quality Control Board that mandated protection of nearby Clear Lake from excessive eutrophication. Two means were available for compliance with the order at that time:

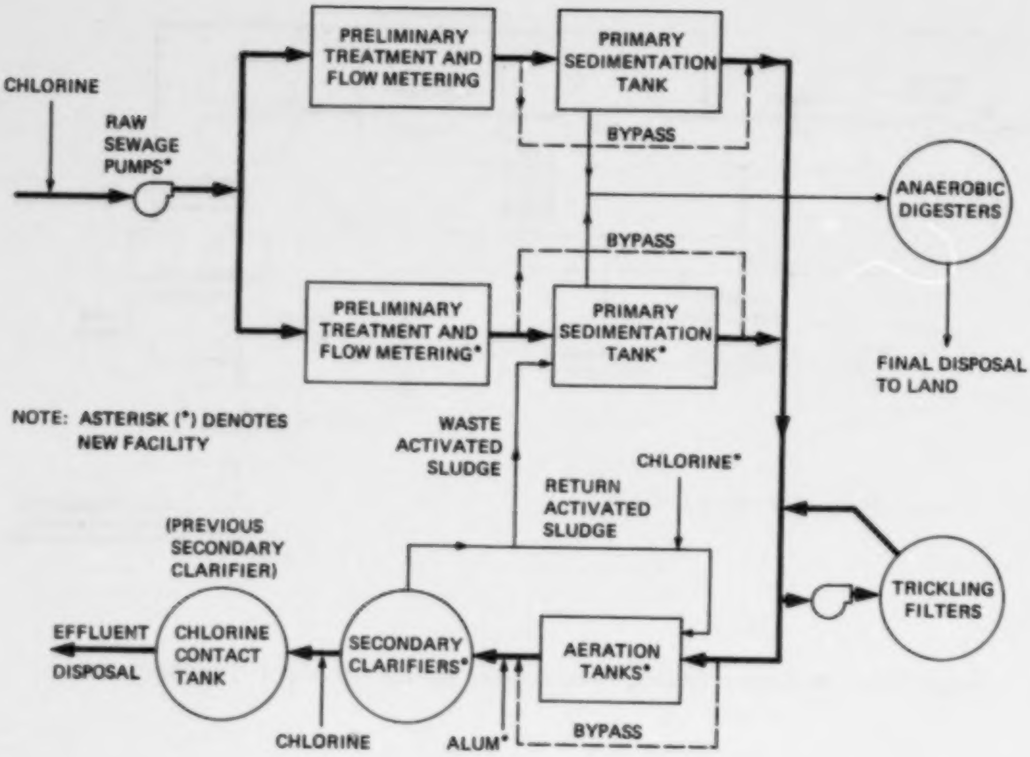
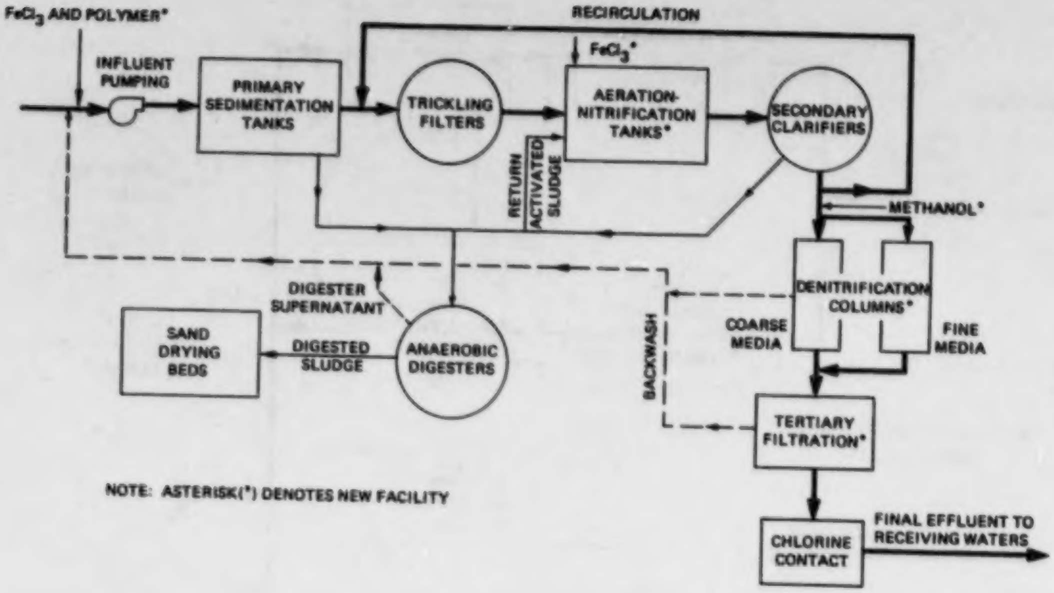

CHLORINE
RAW SEWAGE PUMPS*
PRELIMINARY TREATMENT AND FLOW METERING
PRIMARY SEDIMENTATION TANK
BYPASS
ANAEROBIC DIGESTERS
BYPASS
PRELIMINARY TREATMENT AND FLOW METERING*
PRIMARY SEDIMENTATION TANK*
FINAL DISPOSAL TO LAND
NOTE: ASTERISK (*) DENOTES NEW FACILITY
WASTE ACTIVATED SLUDGE
CHLORINE*
RETURN ACTIVATED SLUDGE
(PREVIOUS SECONDARY CLARIFIER)
TRICKLING FILTERS
EFFLUENT DISPOSAL
CHLORINE CONTACT TANK
SECONDARY CLARIFIERS*
AERATION TANKS*
CHLORINE
ALUM*
BYPASS

Figure 58. Corvallis, Oregon, wastewater treatment plant flow diagram.

(1) export of wastewater from the basin or (2) providing nutrient removal prior to discharge to Clear Lake. The second option was chosen and the District obtained an EPA grant to demonstrate full-scale nitrogen and phosphorus removal.[11]

The original plant consisted of a rock trickling filter plant (actually two plants in parallel) with anaerobic sludge digestion for solids processing. The modified flow sheet incorporating nutrient removal is shown in Figure 59. Added facilities are identified by asterisks and include new aeration tanks for nitrification, denitrification columns, tertiary filtration, and facilities for metal salt, polymer, and methanol addition. All existing structures were incorporated into the upgraded plant. Two separate types of denitrification


FeCl3 AND POLYMER*
INFLUENT PUMPING
PRIMARY SEDIMENTATION TANKS
RECIRCULATION
TRICKLING FILTERS
FeCl3*
AERATION-NITRIFICATION TANKS*
RETURN ACTIVATED SLUDGE
SECONDARY CLARIFIERS
METHANOL*
DENITRIFICATION COLUMNS*
COARSE MEDIA
FINE MEDIA
BACKWASH
DIGESTER SUPERNATANT
SAND DRYING BEDS
DIGESTED SLUDGE
ANAEROBIC DIGESTERS
TERTIARY FILTRATION*
CHLORINE CONTACT
FINAL EFFLUENT TO RECEIVING WATERS

NOTE: ASTERISK(*) DENOTES NEW FACILITY

Figure 59. El Lago, Texas, wastewater treatment plant flow diagram.

columns, with fine and with coarse media, were provided to allow comparison of alternative designs. A minor operational difference from other coupled trickling filter-activated sludge plants discussed in this report is recirculation of clarifier effluent rather than trickling filter effluent to the trickling filter influent line. This results in a reduced organic loading to the filter with a corresponding increase to the aeration tank.

Tables 24 and 25 are tabular summaries of initial performance of the plant and performance 1 yr later, respectively. The improvements in all parameters of effluent quality obtained after a year of operating experience are evident.

The capital costs for the modifications to the El Lago facility were incurred over a 2-yr period (1971-1973) and totaled $312,365, including change orders. This cost includes the provision of dual denitrification facilities; had only one type of denitrification system been included, it is estimated that the total cost would have been about $75,000 less. The only increase in operating costs has been for chemicals and power, which has totaled

TABLE 24. INITIAL PERFORMANCE OF EL LAGO TREATMENT PLANT--JUNE 4 THROUGH JULY 6, 1973 (REF. 11)

Constituent	Raw wastewater[a,b]	Primary influent	Primary effluent	Nitrified effluent	Denitrified effluent[c,d]	Final effluent
Total phosphorus, mg/l	12.8	15.4	8.4	7.3	6.6	4.8
Soluble phosphorus, mg/l	10.3	4.7	4.1	3.4	5.5	3.6
Suspended solids, mg/l	113	289	72	37	17	3
Ammonia nitrogen, mg/l	18.7	21.7	21.5	0.9	0.8	0.6
Total Kjeldahl nitrogen, mg/l	42.6	38.6	30.2	3.7	2.4	3.3
Nitrate nitrogen, mg/l[e]	-	-	-	15.2	2.6	2.3
BOD_5, mg/l	175	222	-	65[f]	9	9
COD, mg/l	297	488	181	121[f]	72	51
Temperature, C	26.5	-	-	-	-	-

[a] Average flow to plant: 0.307 mgd (0.013 m^3/sec)

[b] Peak daily flow to plant: 1.0 mgd (0.044 m^3/sec)

[c] Average flow to denitrification columns: 0.254 (0.011 m^3/sec)

[d] Peak daily flow to denitrification columns: 0.420 mgd (0.018 m^3/sec)

[e] Nitrite nitrogen always less than 0.2 mg/l

[f] Includes methanol

2.5¢/m^3 (9.6¢/1,000 gal). It was found that the plant operators previously employed could adapt to advanced waste treatment processes and that no increase in staff was required.

San Pablo, California

The San Pablo Sanitary District operates a 0.55-m^3/sec (12.5-mgd) coupled trickling filter-activated sludge plant designed for year-round nitrification. The original plant consisted of primary sedimentation with effluent chlorination and digestion for solids processing. Additions completed in 1972 resulted in the plant flow diagram shown in Figure 60. New facilities included additional primary sedimentation tanks, a plastic media roughing trickling filter, aeration-nitrification tanks, secondary clarifiers, an additional chlorine contact tank, dissolved air flotation sludge thickeners, and two additional digesters. The San Pablo plant differs from the other plants discussed in this report in that the previous plant did not include a trickling filter.

TABLE 25. SUBSEQUENT PERFORMANCE OF EL LAGO TREATMENT PLANT--OCTOBER 1 THROUGH OCTOBER 31, 1974 (REF. 11)

Constituent	Primary influent[a,b]	Primary effluent	Nitrified effluent	Denitrified effluent[c,d] Column No. 1	Denitrified effluent[c,d] Column No. 2	Final effluent
Total phosphorus, mg/l[e]	12	3.6	3.1	-	-	0.41
Soluble phosphorus, mg/l	1.8	1.0	0.41	-	-	0.40
Suspended solids, mg/l	296	51	81	31	44	1
Ammonia nitrogen, mg/l	-	18	0.4	-	0.4	0.3
Total Kjeldahl nitrogen, mg/l	-	24	2.6	-	1.5	0.9
Nitrate nitrogen, mg/l	-	-	15	1.9	0.9	0.6
BOD_5, mg/l	-	-	62[f]	16	12	3
COD, mg/l	-	-	113[f]	44	36	19
Temperature, C	-	-	21	-	-	-

[a] Average daily flow to plant: 0.301 mgd (0.013 m^3/sec)

[b] Peak daily flow to plant: 0.47 mgd (0.021 m^3/sec)

[c] Average flow to denitrification columns: 0.282 mgd (0.012 m^3/sec)

[d] Peak daily flow to denitrification columns: 0.470 mgd (0.021 m^3/sec)

[e] Reduction of P due to ferric chloride addition to primary and nitrification step (37 mg/l as Fe). Also polymer, DOW A-23, added to primary at 0.23 mg/l and to tertiary filter at 0.17 mg/l

[f] Includes methanol

Current discharge requirements are essentially those defined by EPA for municipal secondary treatment plants with additional requirements set on effluent toxicity. In addition, the effluent pH is restricted to the range of 6.5 to 8.5. The acid production from nitrification and subsequent chlorination normally forces the effluent pH below 6.5. This has necessitated the addition of caustic to the final effluent to raise the pH above 6.5. Toxicity requirements state that fish bioassays must be run on the undiluted effluent and that 90 percent of a series of 10 consecutive tests must show 70 percent fish survival for 96 hr. Experience at this plant has indicated that the requirements cannot be met without removal of ammonia through nitrification.[12]

A primary design consideration in laying out the plant for nitrification was the presence of a significant volume fraction (11 to 13 percent) of potentially toxic industrial wastes in the influent wastewater. Tank truck washing residues and the waste from a manufacturer of organic peroxide and phenol formaldehyde are the major industrial waste sources. The roughing filter is used in the treatment plant to protect the nitrifying organisms from influent wastewater toxicity. Toxic dumps have caused severe sloughing and loss of growth on the media in the roughing filter, but nitrification remained unaffected.[12]

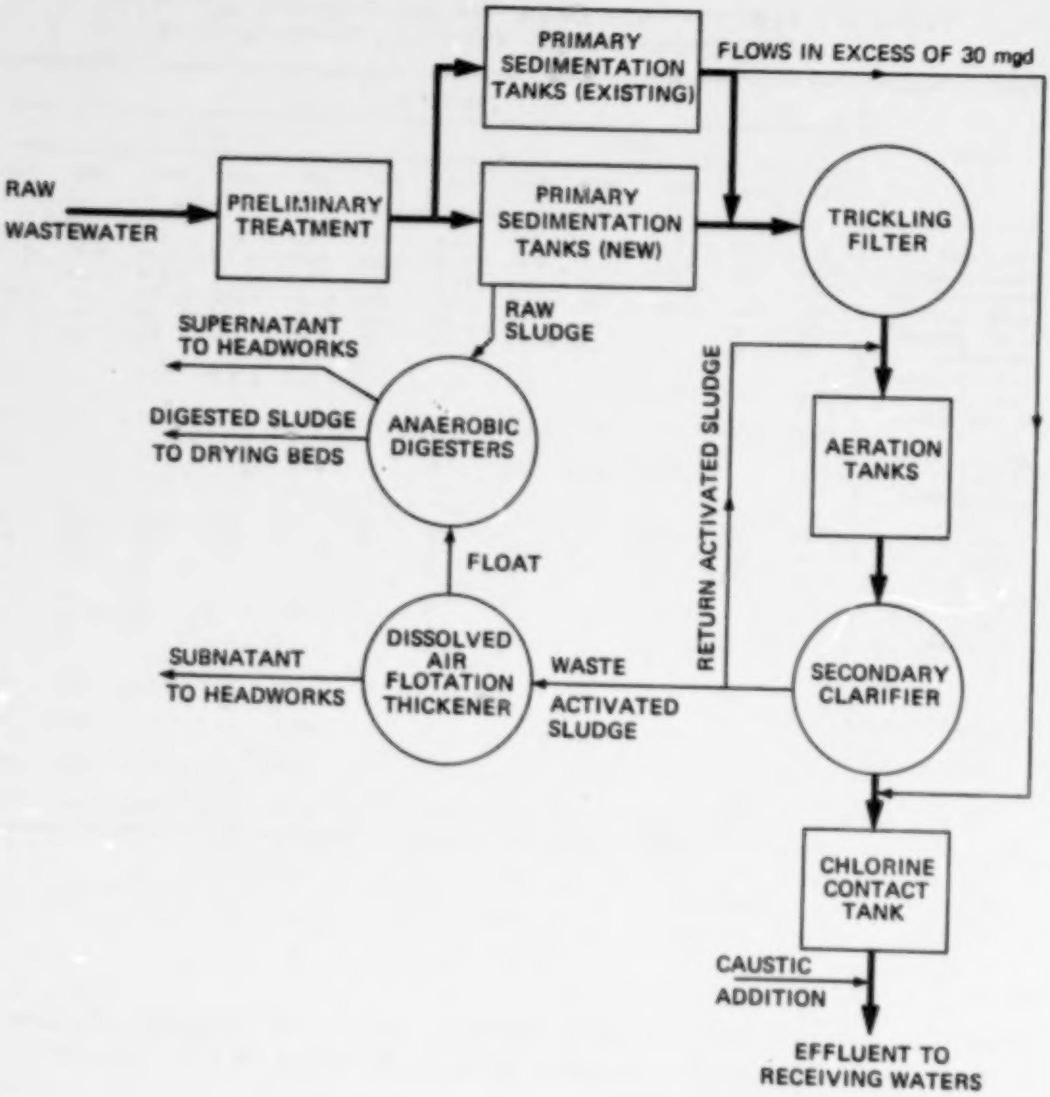

RAW WASTEWATER
PRELIMINARY TREATMENT
PRIMARY SEDIMENTATION TANKS (EXISTING)
FLOWS IN EXCESS OF 30 mgd
PRIMARY SEDIMENTATION TANKS (NEW)
TRICKLING FILTER
RAW SLUDGE
SUPERNATANT TO HEADWORKS
DIGESTED SLUDGE TO DRYING BEDS
ANAEROBIC DIGESTERS
RETURN ACTIVATED SLUDGE
AERATION TANKS
FLOAT
SUBNATANT TO HEADWORKS
DISSOLVED AIR FLOTATION THICKENER
WASTE ACTIVATED SLUDGE
SECONDARY CLARIFIER
CHLORINE CONTACT TANK
CAUSTIC ADDITION
EFFLUENT TO RECEIVING WATERS

Figure 60. San Pablo, California, wastewater treatment plant flow diagram.

Consistent year-round nitrification is obtained in this plant as indicated in Table 26. While only once-weekly and once-monthly analyses of nitrogen are included in Table 26, the consistency of nitrification in the plant is confirmed by daily ammonia nitrogen analyses of grab samples,which normally

TABLE 26. PERFORMANCE DATA FOR SAN PABLO WASTEWATER TREATMENT PLANT--JULY 1973 THROUGH JUNE 1974

Parameter	Year and Month											
	1973						1974					
	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	Mar	April	May	June
Flow, mgd[a]	6.3	5.6	5.7	5.9	9.7	8.4	9.1	6.6	8.3	7.4	6.2	6.2
Return sludge ratio	0.30	0.34	0.25	0.31	0.24	0.29	0.22	0.29	0.19	0.23	0.44	0.44
Temperature, C	22.2	23.6	24.4	23.2	20.3	18.5	17.0	17.8	18.2	19.0	22.0	22.0
MLSS, mg/l	1,400	1,500	1,760	1,770	1,610	1,550	1,480	1,420	1,520	1,500	-	2,830
MLVSS, percent	79	80	76	76	72	73	74	78	74	70	-	78
SVI, ml/gm	73	68	48	66	73	85	87	96	101	69	82	77
Solids retention time, days	12.3	13.9	15.4	11.8	9.1	12.1	7.4	9.2	7.5	6.8	-	6.6
Aeration time, hr	10.8	12.1	11.9	11.5	7.0	6.1	7.5	10.3	8.2	9.2	-	5.5
Air use, million ft^3/day[b]	-	-	20.0	19.4	19.4	20.0	-	-	18.3	-	-	15.3
Roughing filter effluent												
BOD_5, mg/l[c]	121	125	134	131	88	81	91	92	107	111	140	123
COD, mg/l[c]	279	306	283	281	281	212	249	240	314	247	332	327
SS, mg/l[c]	-	97	67	95	59	73	96	79	135	95	130	118
Secondary effluent												
BOD_5, mg/l[d]	16	4	6	6	7	4	3	4	4	5	3	2
COD, mg/l[d]	78	56	55	62	54	59	53	53	61	54	50	46
SS, mg/l[d]	8	7	8	6	3	7	4	4	6	7	4	4
Organic nitrogen, mg/l[e]	7.8	3.8	3.1	3.4	3.4	4.0	4.8	2.2	2.8	4.2	5.1	2.6
Ammonia nitrogen, mg/l[d]	<0.2	<0.2	<0.2	<0.2	0.7	0.1	<0.2	<0.2	<0.2	<0.2	<0.2	0.3
Nitrate nitrogen, mg/l[e]	18.6	19.8	18.2	20.4	24.8	17.6	17.0	15.8	11.8	11.2	10.7	19.0
Nitrite nitrogen, mg/l[d]	0.05	0.02	0.02	0.02	0.26	0.05	0.22	0.02	0.09	0.10	0.06	0.08

[a] mgd x 0.044 = m^3/sec

[b] million ft^3/day x 28,000 = m^3/day

[c] Grab sample at peak flow each weekday

[d] Composite sample, once per week

[e] Composite sample, once per month

show less than 0.2 mg/l ammonia nitrogen. Wastewater temperatures are favorable for nitrification, typically dropping to only 17 C, the average monthly temperature in January.

Construction cost of the added facilities totaled $4.9 million, including legal, engineering, and contract administration costs. The contract was awarded in October 1970, and construction was essentially completed by October 1972. Treatment plant operating costs totaled $397,500 for fiscal year 1973-1974, during which time a total of 9.8 million m^3 (2,600 million gal) were processed through the secondary treatment facilities. On a unit basis, treatment plant operation and maintenance costs totaled 4.0¢/m^3 (15.3¢/1,000 gal) for fiscal year 1973-1974.[12]

Summary of Plant Design Data

A summary of design data for the Livermore, Lompoc, Corvallis, El Lago, and San Pablo treatment plants is presented in Table 27. The basis for design in these cases are essentially empirical, with design changes made to solve problems encountered at previously constructed plants. The next subsection provides a review of nitrification kinetic theory, which can be used as the basis for development of more rational design procedures.

TABLE 27. DESIGN CRITERIA FOR UPGRADED PLANTS

Design parameter	Livermore	Corvallis	El Lago	Lompoc	San Pablo
Flow, mgd[a]					
Before upgrading					
ADWF	2.5	4.0	0.5	1.8	-
After upgrading					
ADWF	5.0	9.7	0.5	5.0	12.5
PDWF	10.0	13.4	-	9.1	-
PWWF	18.0	28.0	-	16.0	30.0
Future ultimate					
ADWF	10.0	17.3	-	-	-
Influent Concentrations, mg/l					
BOD_5	300	135	160	300	340
Suspended solids	300	140	195	300	300
Ammonia nitrogen	-	-	25	25	-
Influent Loadings, lb/day[b]					
BOD_5	12,500	10,000	670	15,500	35,400
Suspended solids	12,500	11,500	810	17,000	31,300
Ammonia nitrogen					
Average	-	-	105	2,100	-
Peak	-	-	-	3,300	-
Primary Treatment					
Assumed BOD_5 removal, percent	35	30	25	35	-
Assumed suspended solids removal, percent	60	65	70	65	-
Trickling Filters					
Number	2	2	2	2	1
Diameter, ft[c]	110	160	-	85	52
Filter media depth, ft[c]	4.25	8	6.5	20[e]	18[e]
Circulation ratio to ADWF	1.5 - 3.0	1.2 - 2.4	1.6	1.5	1.0 - 2.4
Loading, lb BOD_5/1,000 ft^3/day[d]	100	24	12	71.7	350
Assumed BOD_5 removal, percent	50[f]	47[f]	-	43[f]	-

(continued on next page)

TABLE 27. (continued)

Design parameter	Livermore	Corvallis	El Lago	Lompoc	San Pablo
Activated Sludge Aeration Tanks					
Number	1[i]	2	2	2[i]	2
Length per tank, ft[c]	160	156	-	290	252
Width, ft[c]	30	40	-	30	50
Water depth, ft[c]	15	15	-	15	15
Total volume, 1,000 ft^3 [g]	144	187	101	261	378
Detention time, hr, based on ADWF and raw sewage flow	5.2	3.4	6.1	4.7	5.4
MLSS	-	2,000	-	3,050	-
Volumetric loading, lb BOD$_5$/1,000 ft^3/day[d]	26	22	-	35	-
Aeration Blowers					
Number	3	4	2	3	4
Capacity each, ft^3/min[j]	2,000	5,900	450	5,200	6,000
Air supplied, ft^3/(lb BOD$_5$ + lb peak NOD removed)[k]	1,200[l] (BOD$_5$ only)	1,370[l]	-	1,180[l]	-
Secondary Sedimentation Tanks					
Number	1	2[h]	2	3	2
Diameter, ft[c]	90	115	-	65	180x60[m]
Side water depth, ft[c]	12	10	-	16	-
Overflow rate, gpd/ft^2 [n]					
ADWF	787	520	320	503	580
PWWF	2,833	1,500	1,060	1,609	1,390
Activated Sludge Treatment					
Assumed BOD removal, percent	88	80	-	86	-
Assumed suspended solids removal, percent	86	80	-	-	-
Assumed ammonia nitrogen removal, percent	-	-	92	99.6	-
Overall Plant Performance					
Assumed BOD$_5$ removal, percent	96	93	97	95	-
Assumed suspended solids removal, percent	96	93	95	95	-
Assumed ammonia nitrogen removal, percent	-	-	92	99.6	-
Effluent BOD$_5$ concentration, mg/l	15	10	5	15	-
Effluent suspended solids concentration, mg/l	15	10	10	15	-
Effluent ammonia nitrogen concentration, mg/l	-	1	2	0.1	-
Effluent coliform concentration, MPN/100 ml	5.0[o]	<100	-	2.2	-

[a] mgd x 0.044 = m^3/sec

[b] lb/day x 0.454 = kg/day

[c] ft x 0.305 = m

[d] lb/1,000 ft^3/day x 0.016 = kg/m^3/day

[e] Plastic media

[f] Unsettled effluent

[g] 1,000 ft^3 x 28 = m^3

[h] With center-well flocculators

[i] Two passes per tank

[j] ft^3/min x 0.028 = m^3/min

[k] ft^3/lb x 0.0624 = m^3/kg

[l] One blower for standby

[m] Rectangular sedimentation tanks

[n] gpd/ft^2 x 0.041 = m^3/m^2/day

[o] 5-day median discharge requirement; later changed to 2.2 MPN/100 ml as a 7-day median

NITRIFICATION KINETICS IN THE COUPLED TRICKLING FILTER-ACTIVATED SLUDGE PROCESS

The term nitrification is applied to the conversion of ammonia nitrogen in wastewater to nitrate nitrogen through biochemical reactions. Nitrification is often used as a method of ammonia removal, but it can also be used as the first step in a nitrogen removal process, preceding biological denitrification which involves reduction of nitrate to nitrogen gas which escapes to the atmosphere. Nitrate nitrogen can stimulate aquatic growths, and for this reason, removal of all nitrogen compounds may be mandated by regulatory agencies. Ammonia nitrogen, in addition to being a growth stimulant, imposes an oxygen demand on receiving waters, interferes with chlorine disinfection, and is toxic to fish life. Its removal may be specifically required by regulatory agencies, or it may be undertaken in order to meet disinfection or toxicity requirements.

The information presented below is intended to provide background information prior to development of design procedures discussed later. The material is taken principally from the EPA Technology Transfer publication, Process Design Manual for Nitrogen Control.[9] The reader is referred to that manual for a more complete presentation of the concepts discussed briefly here.

Oxidation and Synthesis Reactions

The two principal genera involved in nitrification are Nitrosomonas and Nitrobacter. These groups are classed as autotrophic organisms and are distinguished from heterotrophic organisms in that they derive energy for growth from oxidation of inorganic nitrogen compounds rather than from oxidation of organic waste matter. Each group is limited to the oxidation of a specific species of nitrogen compound. Nitrosomonas can oxidize ammonia to nitrite, and Nitrobacter can complete the reaction by oxidizing nitrite to nitrate. The stoichiometric reactions are as follows:

$$NH_4^+ + 1.5\ O_2 \longrightarrow 2H^+ + H_2O + NO_2^- \qquad (11)$$

$$NO_2^- + 0.5\ O_2 \longrightarrow NO_3^- \qquad (12)$$

The overall oxidation of ammonia is obtained by adding the two equations:

$$NH_4^+ + 2\ O_2 \longrightarrow NO_3^- + 2H^+ + H_2O \qquad (13)$$

These reactions furnish energy required for growth of the nitrifying organisms. Using representative measurements of yields and oxygen consumption for Nitrosomonas and Nitrobacter and assuming that the empirical formula for bacterial cells is $C_5H_7NO_2$, the overall synthesis and oxidation reaction is:

$$NH_4^+ + 1.83\ O_2 + 1.98\ HCO_3^- \longrightarrow 0.021\ C_5H_7NO_2$$
$$+ 1.04\ H_2O + 0.98\ NO_3^- + 1.88\ H_2CO_3 \qquad (14)$$

Two important aspects of the oxidation reaction are oxygen and alkalinity requirements. From Equation 13, 4.57 kg O_2/kg NH_4^+-N are theoretically required for the oxidation of ammonia. From Equation 14, it is evident that synthesis lowers actual oxygen requirements for nitrification to 4.18 kg O_2/kg NH_4^+-N. A value sufficiently accurate to be used in engineering calculations for aeration requirements is 4.6 kg O_2/kg NH_4^+-N.

The oxygen demand for nitrification is significant. About 140 mg/l of oxygen are required to oxidize 30 mg/l of ammonia nitrogen. It is this high demand which has created difficulties over the years in operating the Livermore plant.

Destruction of alkalinity during the oxidation reaction can create problems by lowering the wastewater pH. Approximately 7.1 kg of alkalinity as $CaCO_3$ are destroyed per kg NH_4^+-N oxidized. Thus, over 210 mg/l of alkalinity as $CaCO_3$ will be destroyed during oxidation of 30 mg/l of ammonia nitrogen. In many wastewaters, there is insufficient alkalinity initially present to leave a significant residual for buffering the wastewater during the nitrification process. The significance of pH depression is that nitrification rates decrease as pH level drops. For a full discussion of the effect of pH on nitrification, see the Technology Transfer publication, Design Manual for Nitrogen Control.[9]

Nitrification Kinetics

A description of ammonia and nitrite oxidation can be derived from an examination of the growth kinetics of *Nitrosomonas* and *Nitrobacter*. *Nitrosomonas'* growth is limited by the concentration of ammonia, while *Nitrobacter's* growth is limited by the concentration of nitrite.

The kinetic equation proposed by Monod is used to describe the biological growth kinetics of either *Nitrosomonas* or *Nitrobacter*.

$$\mu = \hat{\mu} \frac{S}{K_s + S} \qquad (15)$$

where: μ = microorganism growth rate, day^{-1},

$\hat{\mu}$ = maximum microorganism growth rate, day^{-1},

S = growth limiting substrate concentration, mg/l, and

K_s = half velocity constant = substrate concentration at half the maximum growth rate, mg/l.

Since the maximum growth rate of *Nitrobacter* is considerably larger than that of *Nitrosomonas*, the latter representing conversion of ammonia to nitrite, the *Nitrosomonas* reaction is the rate limiting step.

The ammonia oxidation rate, q_N, can be related to the *Nitrosomonas* growth rate as follows:

$$q_N = \frac{\mu_N}{Y_N} = \hat{q}_N \frac{N}{K_N + N} \tag{16}$$

where: μ_N = *Nitrosomonas* growth rate, day^{-1},

$\hat{\mu}_N$ = peak *Nitrosomonas* growth rate, day^{-1},

q_N = ammonia oxidation rate, kg NH_4^+-N oxidized/kg *Nitrosomonas*/day,

$\hat{q}_N = \frac{\hat{\mu}_N}{Y_N}$ = peak ammonia oxidation rate, kg NH_4^+-N oxidized/kg *Nitrosomonas*/day,

Y_N = organism yield coefficient, kg *Nitrosomonas* grown/kg NH_4^+-N removed,

N = NH_4^+-N concentration, mg/l, and

K_N = half-saturation constant, mg/l NH_4^+-N.

The nitrifier growth rate can be utilized in the design of activated sludge systems by noting its inverse relationship to the system solids retention time:

$$\theta_{c_N} = \frac{1}{\mu_N} \tag{17}$$

where θ_{c_N} = nitrifier solids retention time, days. θ_{c_N} can be calculated from system operating data by dividing the total amount (or inventory) of microbial mass in the treatment system by the quantity wasted daily.

The nitrifier growth rate, μ_N, is affected by factors other than ammonia nitrogen concentration, principally wastewater temperature, dissolved oxygen concentration, and pH. An expression relating μ_N to all these parameters can be developed:

$$\mu_N = 0.47 \left[e^{0.096\,(T - 15)}\right]\left[1 - 0.833\,(7.2 - pH)\right] \quad (18)$$

$$\times \left[\frac{N}{N + 10^{0.051\,T - 1.158}}\right]\left[\frac{DO}{DO + 1.3}\right]$$

where: T = temperature, C, and
DO = dissolved oxygen, mg/l.

The first term in brackets indicates that the nitrifier growth rate decreases with a drop in temperature. This is a well-known phenomenon and must be taken into account when designing nitrification systems for cold weather regions. This relation is valid for temperatures between 8 and 30 C. The second term in brackets describes the effect of pH on nitrifier growth rate. The term is taken as unity for pH values above 7.2. The third and fourth terms are the Monod expressions relating ammonia nitrogen and dissolved oxygen concentrations, respectively, to growth rate. The inhibiting effect of low DO levels at Livermore was discussed in Section 8.

In a combined, single-stage, carbon oxidation-nitrification process, sufficient organic carbon is normally available in the reactor influent to sustain a flourishing growth of heterotrophic bacteria. In a separate-stage nitrification reactor, organic carbon availability is reduced but usually still adequate to stimulate some heterotrophic organism growth. Since the yield coefficient for heterotrophic baceterial growth is far greater than for autotrophic nitrifier growth, it is possible for the growth rate of the heterotrophs to be established at a level exceeding the maximum possible growth rate of the nitrifying organisms. This may occur in both combined carbon oxidation-nitrification processes and in separate-stage nitrification and depends on the relative substrate concentrations of organic carbon and ammonia and on the nitrifier solids retention time. When this occurs, the slower growing nitrifiers will gradually diminish in proportion to the total population and be washed out of the system.

Thus, for consistent nitrification to occur, the following condition must be satisfied:

$$\dot{\mu}_N \geq \mu_H \quad (19)$$

where: $\dot{\mu}_N$ = maximum possible nitrifier growth rate under environmental conditions of T, pH, DO, and $N >> K_N$, and
μ_H = net growth rate of the heterotrophic population.

$\dot{\mu}_N$ is related to μ_N in the following manner:

$$\mu_N = \dot{\mu}_N \left[\frac{N}{N + 10^{0.051\,T - 1.158}} \right] \tag{20}$$

where: $$\dot{\mu}_N = 0.47 \left[e^{0.098\,(T-15)} \right] \left[1 - 0.833\,(7.2 - \text{pH}) \right] \left[\frac{DO}{DO + 1.3} \right] \tag{21}$$

Equation 19 is usually expressed in reciprocal form in terms of solids retention times:

$$\theta^d_{c_N} \geq \theta^m_{c_N} \tag{22}$$

where: $\theta^d_{c_N}$ = design nitrifier solids retention time, days, and

$\theta^m_{c_N}$ = minimum nitrifier solids retention time, days, for nitrification at given pH, temperature, and DO levels.

Because $\theta^m_{c_N}$ is fixed by environmental conditions, Equation 22 is satisfied by modifying $\theta^d_{c_N}$. The various ways of meeting this criterion can be established by examining the following growth relationship for the heterotrophic population.

$$\mu_H = \frac{1}{\theta^d_{c_H}} = Y_H q_H - K_d \tag{23}$$

where: $\theta^d_{c_H}$ = design heterotroph solids retention time, days,

Y_H = heterotroph yield coefficient, kg VSS grown/kg substrate (BOD_5) removed,

q_H = substrate removal rate, kg BOD_5 removed/kg VSS/day, and

K_d = heterotroph decay coefficient, day^{-1}.

The substrate removal rate is defined:

$$q_H = \frac{S_1 - S_2}{X_{H_{ML}}\,t} \tag{24}$$

where: S_1 = aeration tank influent total BOD_5, mg/l,

S_2 = aeration tank effluent soluble BOD_5, mg/l,

t = nominal hydraulic detention time (or aeration time) based on Q, days, and

$X_{H_{ML}}$ = MLVSS, mg/l.

Since both Y_H and K_d are assumed to be constant, $\theta^d_{c_H}$ can only be altered by manipulating q_H; $\theta^d_{c_H}$ will be increased by reducing q_H. One way of doing this is by reducing S_1 through placing an organic carbon removal step ahead of the nitrification stage, creating a separate-stage nitrification process. This function is performed at Livermore by the trickling filter ahead of the activated sludge process. Solids retention times, $\theta^d_{c_H}$, for separate-stage nitrification processes can be 20 days or longer.

Another procedure for reducing the substrate removal rate is to increase the quantity of biological solids in the system. This can be done by increasing the biological solids concentration (MLVSS) in the aeration tank or by increasing the aeration tank volume.

In most nitrification systems, the level set for heterotroph solids retention time, $\theta^d_{c_H}$, establishes the nitrifier solids retention time, $\theta^d_{c_N}$. The coupled trickling filter-activated sludge system is an important exception, as noted in Section 8 and discussed further below.

With specific values of $\theta^d_{c_N}$, T, pH, and DO, Equation 20 can be solved for the effluent ammonia level. Figure 61 was developed through such a procedure, assuming T = 15 C, DO = 2 mg/l, and $7.2 < pH < 8$. The principal causes of the sharpness of the ammonia breakthrough are the low value of K_N (0.4 mg/l $NH_4^+ - N$ in this case) and the fact that the figure represents steady state conditions. Under normal diurnal load variations, removal would increase less rapidly with increasing solids retention time.

Lawrence and McCarty[13] have introduced the concept of a safety factor (SF) in the application of biological treatment process kinetics to design. Their safety factor is defined as the ratio of the design solids retention time to the minimum solids retention time, and it can also be related to nitrifier growth rates:

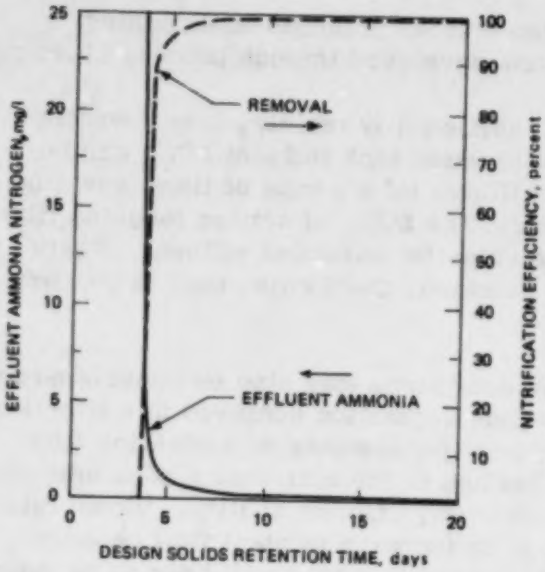


Figure 61. Effect of solids retention time on effluent ammonia concentration and nitrification efficiency (Ref. 9).

$$SF = \frac{\theta^{d}_{c_N}}{\theta^{m}_{c_N}} = \frac{\dot{\mu}_N}{\mu_N} \qquad (25)$$

A conservative safety factor was recommended to minimize process variations caused by pH extremes, low DO concentrations, and toxicants. The safety factor can also be used to ensure that ammonia breakthrough does not occur during diurnal peaks in load. A normal range for safety factors is 2.0 to 5.0.

The relationships developed above will be used to compute aeration tank size requirements in the following subsection.

DESIGN CONSIDERATIONS

Principal factors to be considered in design of a coupled trickling filter-activated sludge plant are aeration tank size and air requirements. Other considerations include reliability, flexibility, use of existing facilities, and expected effluent quality. The procedure developed below for aeration tank sizing is based on the nitrification process kinetics discussed above. The information on air requirements is adopted from the Process Design Manual for Nitrogen Control.[9] The discussions on physical design and expected effluent quality are based principally on the coupled trickling filter-activated sludge plants which have been designed and constructed.

Aeration Tank Size

Difficulties are encountered in computing the required aeration tank size for a coupled trickling filter-activated sludge process because little is known about the characteristics of unsettled trickling filter effluent. In particular, filter effluent contains both waste material which has passed through the roughing filter unoxidized as well as biological solids sloughed from the filter media surface. These solids will be measured in a BOD_5 test although they do not represent substrate material for the activated sludge process in the usual sense. The lack of knowledge also creates problems in assigning values to the coefficients in the equations developed through process kinetics.

Because these sloughed solids will settle fairly rapidly, it is possible that a more accurate characterization of aeration tank influent BOD_5 can be obtained by settling the roughing filter effluent for a period of time, say 1 hr. In Section 8 it was noted that at Livermore, the BOD_5 of settled roughing filter effluent was approximately one-half the value for unsettled effluent. Tests conducted with plastic media filters at Stockton, California, tend to confirm the 50 percent figure.

One hour's settling under quiescent conditions may also be considered to reasonably approximate the degree of solids separation achieved in a trickling filter secondary clarifier. Thus, where past performance of a trickling filter plant is known, the projected influent loading to the activated sludge unit can be based on historical trickling filter secondary effluent quality. Where future filter loadings will be greater because of an increase in plant flow capacity, estimates of the future BOD_5 loading to the aeration tank will have to be made.

Design Procedure--

Expressions for the heterotrophic organism and nitrifier solids retention times were derived in Section 8 as Equations 4 and 6, respectively:

$$\theta_{c_H} = \frac{VX_{H_{ML}}}{QX_{H_2} + Q_W X_{H_W} - QX_{H_1}} \tag{4}$$

and

$$\theta_{c_N} = \frac{VX_{H_{ML}}}{QX_{H_2} + Q_W X_{H_W}} \tag{6}$$

From Equation 23, the inverse of the heterotroph solids retention time is:

$$\frac{1}{\theta_{c_H}} = Y_H q_H - K_d \tag{23}$$

which, when combined with Equation 24, becomes:

$$\frac{1}{\theta_{c_H}} = \frac{Y_H (S_1 - S_2)}{X_{H_{ML}} \quad t} - K_d \tag{26}$$

where S_1 is the BOD_5 of the settled trickling filter effluent.

Combining Equations 4, 6, 23,and 26 yields:

$$\frac{1}{\theta_{c_H}} = \frac{1}{\theta_{c_N}} - \frac{QX_{H_1}}{VX_{H_{ML}}} = \frac{Y_H (S_1 - S_2)}{X_{H_{ML}} t} - K_d \tag{27}$$

Substituting t for $\frac{V}{Q}$ and rearranging terms gives:

$$\frac{1}{\theta_{c_N}} + K_d = \frac{X_{H_1}}{X_{H_{ML}} t} + \frac{Y_H (S_1 - S_2)}{X_{H_{ML}} t} \tag{28}$$

which solving for the nominal aeration time, t, becomes:

$$t = \frac{Y_H (S_1 - S_2) + X_{H_1}}{X_{H_{ML}} \left[\frac{1}{\theta_{c_N}} + K_d \right]} \tag{29}$$

The difficulties in applying Equation 29 are principally related to determining values for S_1, K_d, and X_{H_1}. S_1 has been assumed to be defined by the BOD_5 of the trickling filter effluent setted for 1 hr, but a more accurate determination needs to be developed. K_d is the heterotroph decay coefficient. A value commonly cited is 0.05 day^{-1}, but organisms entering an aeration tank from a trickling filter may have a higher decay rate. X_{H_1} is the concentration of heterotrophic organisms entering the aeration tank. This is difficult to measure because non-microbial volatile suspended solids are also present in the trickling filter effluent and volatile suspended solids are normally taken as representing the microorganisms. The procedure used herein is to apply a yield coefficient, Y_{TF}, to the BOD_5 removed in the trickling filter as follows:

$$X_{H_1} = Y_{TF} (S_o - S_1) \tag{30}$$

where: S_o = trickling filter influent total BOD_5, mg/l.

An estimate based on limited data indicates that $Y_{TF} = 0.5$. This is uncertain, however, and a more accurate estimate is needed. Because Y_{TF} is a "net" yield coefficient, i.e., it includes the effect of organism death, it can be expected to be less than Y_H, which does not include the effect of organism death.

Substituting Equation 30 into Equation 29 provides an expression that can be used to compute the required nominal aeration time for nitrification in an activated sludge system preceded by a roughing filter without intermediate clarification:

$$t = \frac{Y_H (S_1 - S_2) + Y_{TF} (S_o - S_1)}{X_{H_{ML}} \left[\frac{1}{\theta_{c_N}} + K_d \right]} \qquad (31)$$

For comparison, Equation 31 for an activated sludge process not preceded by a roughing filter or one preceded by a roughing filter and intermediate clarification $\left(\text{where } X_{H_1} = 0 \text{ and } \theta_{c_N} = \theta_{c_H} \right)$ reduces to:[9]

$$t = \frac{Y_H (S_1 - S_2)}{X_{H_{ML}} \left[\frac{1}{\theta_{c_N}} + K_d \right]} \qquad (32)$$

Equations 31 and 32 will be utilized in the following design examples to determine aeration times for several hypothetical plant flow schemes.

Design Examples--

Consider a 0.44-m^3/sec (10-mgd) conventional trickling filter plant which is to be upgraded to a coupled trickling filter-activated sludge facility. Assume that the primary effluent BOD$_5$ concentration is 160 mg/l and that the two 46-m (150-ft) dia, 1.8-m (6-ft) deep, rock media filters (total volume, 6,000 m^3 or 212,000 ft^3) produce an effluent with an unsettled BOD$_5$ of 120 mg/l and a settled BOD$_5$ of 60 mg/l. Further assume that the ammonia nitrogen concentration in the trickling filter effluent is 25 mg/l and that the portion of the organic nitrogen not assimilated into biomass or associated with refractory organics is 10 mg/l.

Process kinetics design calculations are given below for three cases: (1) a coupled trickling filter-activated sludge process (i.e., without intermediate clarification), (2) an activated sludge process only, and (3) a trickling filter-activated sludge process with intermediate clarification. In addition, an empirical design calculation using loading factors such as employed for Livermore, Lompoc, and Corvallis will be presented.

Coupled trickling filter - activated sludge process--The design procedure utilized in the Process Design Manual for Nitrogen Control[9] for computing aeration tank size is as follows:

1. Establish the safety factor, SF. Assume SF = 2.5 is required due to transient loading conditions at this particular plant.

2. Calculate the maximum nitrifier growth rate, $\dot{\mu}_N$, at the relevant temperature, dissolved oxygen, and pH. Using Equation 21 and assuming T = 15 C, DO = 2 mg/l, and pH 7.2, results in a value for $\dot{\mu}_N$ of 0.285 days^{-1}.

3. Calculate the minimum nitrifier solids retention time, $\theta_{c_N}^{m}$.

$$\theta_{c_N}^{m} = \frac{1}{\dot{\mu}_N} = \frac{1}{0.285} = 3.51 \text{ days} \tag{33}$$

4. Calculate the design nitrifier solids retention time, $\theta_{c_N}^{d}$.

$$\theta_{c_N}^{d} = SF \times \theta_{c_N}^{m} = = 2.5\ (3.51) = 8.78 \text{ days} \tag{25}$$

5. Calculate the design nitrifier growth rate, μ_N.

$$\mu_N = \frac{1}{\theta_{c_N}^{d}} = \frac{1}{8.78} = 0.114 \text{ days}^{-1} \tag{34}$$

6. Calculate the half-saturation constant, K_N, for ammonia oxidation at 15 C. The proper expression is:

$$K_N = 10^{0.051\,T - 1.158} \tag{35}$$

where T = temperature, C. For this example,

$$K_N = 10^{-0.393} = 0.40 \text{ mg/l}$$

7. Calculate the steady state ammonia nitrogen content of the effluent. For plug flow, the Monod substrate removal rate equation must be integrated over the period of time an element of liquid remains in the aeration tank.[13]

$$\frac{1}{\theta_{c_N}^{d}} = \frac{\dot{\mu}_N (N_o - N_1)}{(N_o - N_1) + K_N \ln \frac{N_o}{N_1}} \quad \text{for } r < 1 \tag{36}$$

where r = return sludge rate,

N_o = influent TKN (ammonia nitrogen plus organic nitrogen), and

N_1 = effluent ammonia nitrogen.

Solving Equation 36 using 35 mg/l for N_O produces a value for $N_1 < 0.1$ mg/l. Transient loading effects will increase this value and are discussed in the Process Design Manual for Nitrogen Control.[9]

8. Calculate the nominal aeration time, t. From Equation 31:

$$t = \frac{Y_H (S_1 - S_2) + Y_{TF} (S_o - S_1)}{Y_{H_{ML}} \left[\frac{1}{\theta^d_{c_N}} + K_d \right]} \tag{31}$$

Substituting appropriate values into the equation:

S_o = 160 mg/l,

S_1 = 60 mg/l,

$S_2 \cong$ 0 mg/l,

Y_H = 0.60 kg VSS/kg BOD_5 removed,

Y_{TF} = 0.50 kg VSS/kg BOD_5 removed,

$X_{H_{ML}}$ = 1,800 mg/l,

$\theta^d_{c_N}$ = 8.78 days, and

K_d = 0.07 day^{-1}:

$$t = \frac{0.6 (60) + 0.5 (160 - 60)}{1,800 \left(\frac{1}{8.78} + 0.07 \right)}$$

$$t = \underline{\underline{0.26 \text{ day}}} = \underline{\underline{6.24 \text{ hr}}}$$

$$V = \underline{\underline{9,860 \text{ m}^3}} \quad \underline{\underline{(348,000 \text{ ft}^3)}}$$

For the above example, a decay coefficient of 0.07 day^{-1} was used. This is slightly higher than the normally cited value of 0.05 and was chosen because a higher decay rate is believed to be appropriate for those circumstances where sloughed organisms enter the aeration tank. Studies involving the contact stabilization process, which may be considered somewhat similar to the coupled trickling filtration-activated sludge process, have indicated such a value for the decay rate.[14]

Activated sludge process only--The procedure for computing aeration tank size for the activated sludge process alone is the same as for the coupled process through Step 7 above. Nominal aeration time is then computed from Equation 32:

$$t = \frac{Y_H (S_1 - S_2)}{X_{H_{ML}} \left[\frac{1}{\theta^d_{c_N}} + K_d \right]} \qquad (32)$$

For this example, K_d will be reduced to 0.05 day^{-1}. Other parameters have the same values as in the previous example.

$$t = \frac{0.6\ (160)}{1{,}800 \left[\frac{1}{8.78} + 0.05 \right]}$$

$$t = \underline{\underline{0.33 \text{ days}}} = \underline{\underline{7.92 \text{ hr}}}$$

$$V = \underline{\underline{12{,}500 \text{ m}^3}} \ \underline{\underline{(441{,}000 \text{ ft}^3)}}$$

This is more than 25 percent greater than the nominal aeration time obtained for the coupled trickling filter-activated sludge process. The increase is caused by Y_H being greater than Y_{TF} and by the higher value of K_d chosen for the former example.

The same safety factor, 2.5, has been used for both examples. The presence of toxic wastes would favor the use of a higher safety factor, say 3.5, in the absence of a roughing filter. In this latter case, the difference in nominal aeration times between the two examples would be greater. For the activated sludge process without a roughing filter:

$$\theta^d_{c_N} = SF \times \theta^m_{c_N} = 3.5\ (3.51) = 12.3 \text{ days}$$

$$t = \frac{0.6\ (160)}{1{,}800 \left(\frac{1}{12.3} + 0.5 \right)}$$

$$t = \underline{\underline{0.41\ \text{day}}} = \underline{\underline{9.84\ \text{hr}}}$$

$$V = \underline{\underline{15{,}520\ \text{m}^3}} \quad \underline{\underline{(548{,}000\ \text{ft}^3)}}$$

This is more than 55 percent larger than for a coupled trickling filter-activated sludge process with a safety factor of 2.5.

Such differences in tank size will not be realized in all cases. The primary advantage of a roughing filter ahead of the activated sludge process is the stability in overall performance which it provides.

Trickling filter-activated sludge process with intermediate clarification--Equation 29 indicates that organisms entering the activated sludge process from the trickling filter influence the aeration time required. If an intermediate clarifier is used to remove the sloughed material, aeration times can be reduced. Aeration time would then be calculated using Equation 28 with $X_{H_1} = 0$, as previously derived in Equation 32.

$$t = \frac{Y_H\ (S_1 - S_2)}{Y_{H_{ML}}\left(\frac{1}{\theta^d_{c_N}} + K_d\right)} \tag{32}$$

$$t = \frac{0.6\ (60)}{1{,}800\left(\frac{1}{8.78} + 0.05\right)}$$

$$t = \underline{\underline{0.12\ \text{day}}} = \underline{\underline{2.88\ \text{hr}}}$$

$$V = \underline{\underline{4{,}660\ \text{m}^3}} \quad \underline{\underline{(161{,}000\ \text{ft}^3)}}$$

This is approximately 55 percent smaller than that calculated for the coupled process without the intermediate clarifier.

Empirical design--In discussions on the designs of the Livermore, Corvallis, and Lompoc plants, it was noted that a loading parameter used in these cases was kg BOD_5/m^3 (lb/1,000 ft^3) trickling filter plus aeration tank volume/day. It is instructive to compare aeration tank sizes computed using such an approach with those calculated using the procedures developed above.

Using an overall loading of 0.4 kg/m^3/day (25 lb/1,000 ft^3/day) requires a total biological treatment volume of 15,120 m^3 (534,000 ft^3) for the 0.44-m^3/sec (10-mgd) flow and a primary effluent BOD_5 of 160 mg/l. Subtracting the filter volume of 6,000 m^3 (212,000 ft^3) gives an aeration tank volume of 9,120 m^3 (322,000 ft^3), which corresponds to an aeration time of 5.78 hr, compared with 6.24 hr calculated using Equation 31.

Aeration Requirements

Care must be exercised in designing aeration systems for activated sludge nitrification. Unlike BOD_5, ammonia is not adsorbed to the biological floc for later oxidation and, therefore, must be oxidized during the relatively short period it is in the nitrification reactor. Sufficient oxygen must be provided to handle the load impressed on the nitrification process at all times.

The diurnal variation in nitrogen loading is very significantly affected by in-process flow equalization and by equalization in the wastewater collection system. Large collection systems serving spread-out urban areas have high built-in storage providing unintentional flow and loading equalization. This relationship is indicated in Figure 62 where the nitrogen load peaking (expressed as the ratio of the maximum hourly load to average load) is plotted for eight treatment plants having no significant in-process equalization. In large plants, such as the Blue Plains plant at Washington, D.C., and Sacramento, California, a spread-out collection system causes moderation of both flow and nitrogen load peaking. In the smaller systems, however, without such flow equalization, ammonia load peaking can be substantial; for example, at the Central Contra Costa (California) Sanitary District's (CCCSD) plant, an hourly peaking factor of 2.4 has been measured. The aeration system must accommodate these changes in loads to avoid ammonia bleed-through during the peak-load period. An early decision must be made during the design process as to what level of peaking of oxygen-demanding substances will be designed for. In addition to peaking of ammonia or organic nitrogen, a concurrent peak may also occur in the loading of organic substances. If very low levels of ammonia nitrogen are required at all times, care must be used to develop a statistical base whereby the frequency of peak oxygen loads can be identified. Not only should average daily peaks be identified, but possibly those occurring on weekly or monthly bases. The additional aeration capacity required for handling diurnal variations in nitrogen load, coupled with the extra tankage and equipment required, may dictate in-plant flow equalization in many instances. The reductions in capital and operating cost of aeration tankage and aeration facilities must be compared with the cost of flow equalization to determine applicability to specific cases. Design procedures for flow equalization are contained in Chapter 3 of the Process Design Manual for Upgrading Existing Wastewater Treatment Plants.[15]

Oxygen Transfer Requirements--

Using diffused air aeration, air rates can be easily modulated to closely match the incoming oxygen-demanding load merely by turning down or shutting

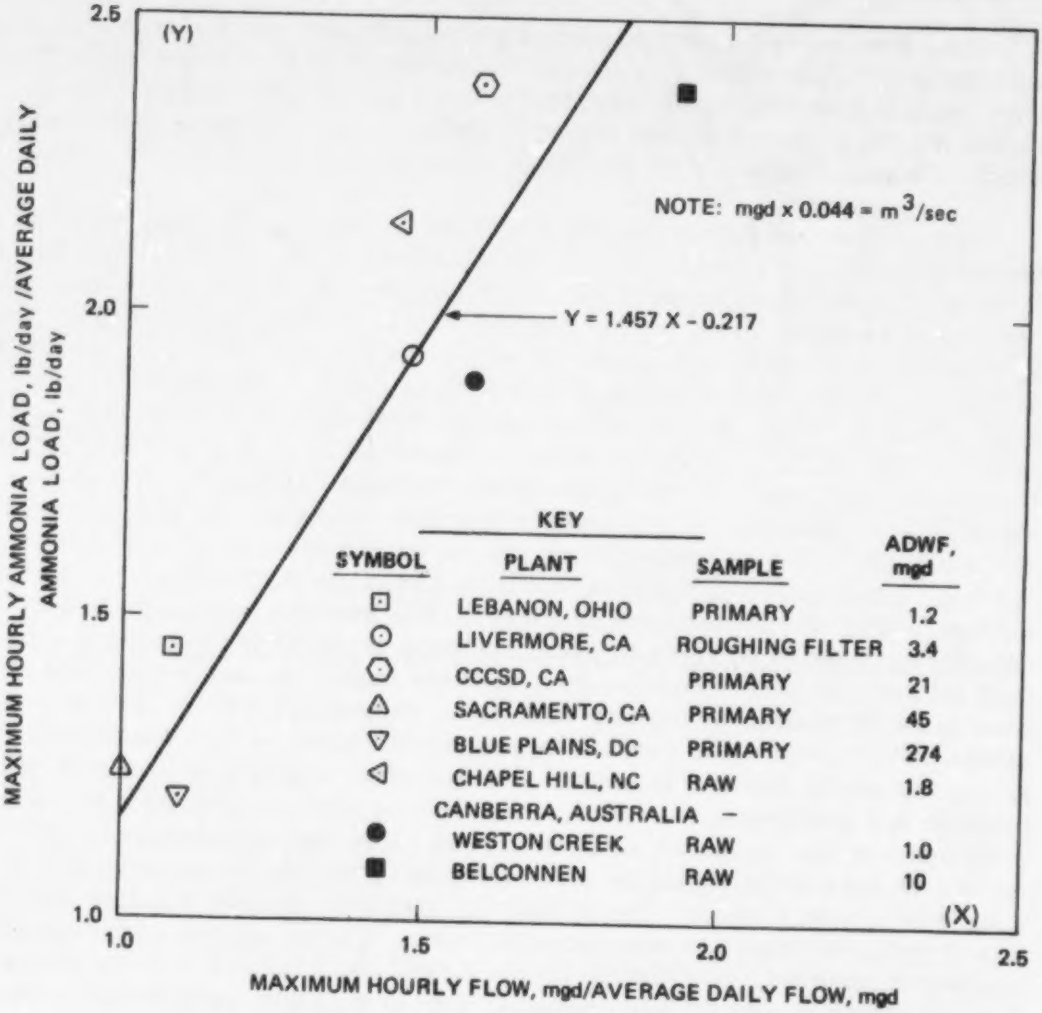


Figure 62. Relation between ammonia nitrogen peaking and hydraulic peaking for treatment plants with no in-process flow equalization (Ref. 9).

off individual blowers. Thus, the diurnal load variations can be matched without the necessity of overaerating the mixed liquor and wasting power as is the case with mechanical surface aerators. Fine bubble diffusers can be arranged across the tank floor, allowing fairly even distribution of energy input. Also, gentler mixing is provided than with mechanical aeration plants, providing less tendency for floc breakup.

Oxygen requirements in all practical cases are compounded by the oxygen required for stabilization of organics. Reasonably exact expressions for oxygen requirements for heterotrophic organisms and nitrifiers have been developed.[14] The approach, however, requires pilot plant data to provide COD balances and sludge yields. In general, this information is not available and a simpler approach may be adopted.[9]

In normal activated sludge treatment when nitrification is not required, the amount of oxygen needed to oxidize the BOD_5 can be calculated by the following equation:

$$B = X\,(BOD_5) \tag{37}$$

where: B = oxygen required for carbonaceous oxidation, mg/l, and
X = a coefficient.

The coefficient X relates to the amount of endogenous respiration taking place and to the type of waste being treated. For normal municipal wastewater, the X value would range from 0.5-0.7 for high-rate activated sludge systems to 1.5 for extended aeration. For conventional activated sludge systems, X can be taken as 1.0.

For nitrification systems, the oxygen requirement for oxidizing ammonia must be added to the requirement for BOD_5 removal. The coefficient for ammonia oxidation can be conservatively taken as 4.6 times the TKN content of the influent to obtain the nitrogen oxygen demand (NOD). (In actual fact, some of the influent nitrogen will be assimilated into the biomass or is associated with refractory organics and will not be oxidized.) Using a value of 1.0 for X in Equation 36, the total oxygen demand in a nitrification system can be approximated as follows:

$$TOD = BOD_5 + NOD \tag{38}$$

where: TOD = the total oxygen demand, mg/l, and
NOD = oxygen required to oxidize the TKN, taken as 4.6 times the influent TKN, mg/l.

Since aeration devices are rated using tap water at standard conditions, the rated performance of the aerator must be converted to actual process conditions by the application of temperature corrections and by factors designated α and β which relate wastewater characteristics to tap water characteristics.

Temperature corrections are made by the relationship:

$$1.024^{(T - 20)}$$

where T = process temperature, C.

The α factor is the ratio of oxygen transfer in wastewater to that in tap water and is represented by the following:

$$\alpha = \frac{K_L \text{ a (process conditions)}}{K_L \text{ a (standard conditions)}} \qquad (39)$$

Values of α can vary widely in industrial waste treatment applications, but for most municipal plants, it will range from 0.40 to 0.90.

The β factor is the ratio of oxygen saturation in wastewater to that in tap water at the same temperature. A value of 0.95 is commonly used. Thus, the amount of oxygen that will actually be transferred under process conditions (W) can be estimated from the amount transferred under test conditions (W_o) by the equation:

$$W = W_o \, \alpha \, (1.024)^{T - 20} \, \frac{\beta C_s - C_1}{9.2} \qquad (40)$$

where: W = oxygen transferred at process conditions, kg/day (lb/day),

W_o = oxygen transferred at standard conditions (T = 20 C, DO $\simeq$ 0, barometric pressure = 760 mm Hg, tap water), kg/day (lb/day),

T = process temperature, C,

C_s = oxygen saturation concentration in water at temperature T, mg/l, and

C_1 = process DO level, mg/l.

The process DO level, C_1, must be set high enough to prevent inhibition of nitrification rates. For this purpose, a minimum value of 2.0 mg/l is recommended. This value is also applicable under peak diurnal load conditions, and the practice of allowing the DO to drop below 2.0 mg/l under peak load is not recommended. If mixed liquor DO is allowed to drop below the minimum during peak load conditions, excessive bleed-through of ammonia can be expected.

Using typical values for domestic sewage ($\alpha = 0.9, \beta = 0.95, C_1 = 2.0$ mg/l, T = 20 C, and $C_s = 9.2$ mg/l) in Equation 40, the relationship between oxygen transfer capability under standard test conditions and under actual process conditions reduces to:

$$W = 0.67\ W_o\text{, or} \tag{41}$$

$$W_o = 1.5\ W \tag{42}$$

The value obtained for W_o can then be used to project horsepower (hp) requirements for mechanical aerators or required volumetric air flow rates for diffused air plants. The latter is accomplished by employing the equation:

$$\begin{aligned} Q_A &= W_o \times \frac{100}{23} \times \frac{1}{1.2} \times \frac{1}{1440} \times \frac{100}{e_o} \\ &= 0.25\ \frac{W_o}{e_o}{}^{*} \end{aligned} \tag{43}$$

where: Q_A = air flow, m^3/min,

e_o = aerator rated oxygen transfer efficiency at standard conditions, percent,

air composition = 23 percent oxygen (weight basis), and

air density = 1.2 kg/m^3

Applying Equations 42 and 43 and the numbers in the previously stated example to diffusers of various efficiencies produces air rates of from 90 to 36 m^3/(kg BOD_5 + kg NOD) (1,450 to 580 ft^3/lb) corresponding to standard-condition diffuser efficiencies of 6 to 15 percent as shown in Figure 63.

Diffuser placement within the aeration tank has been shown in full-scale tests to have a large effect on oxygen transfer efficiency.[16] For oxygen transfer efficiencies, diffusion system design, and layout, the reader should refer to the literature as well as to air diffusion equipment manufacturers.

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*$Q_A = 4.02\ \frac{W_o}{e_o}$, Q_A in ft^3/min and W_o in lb/day

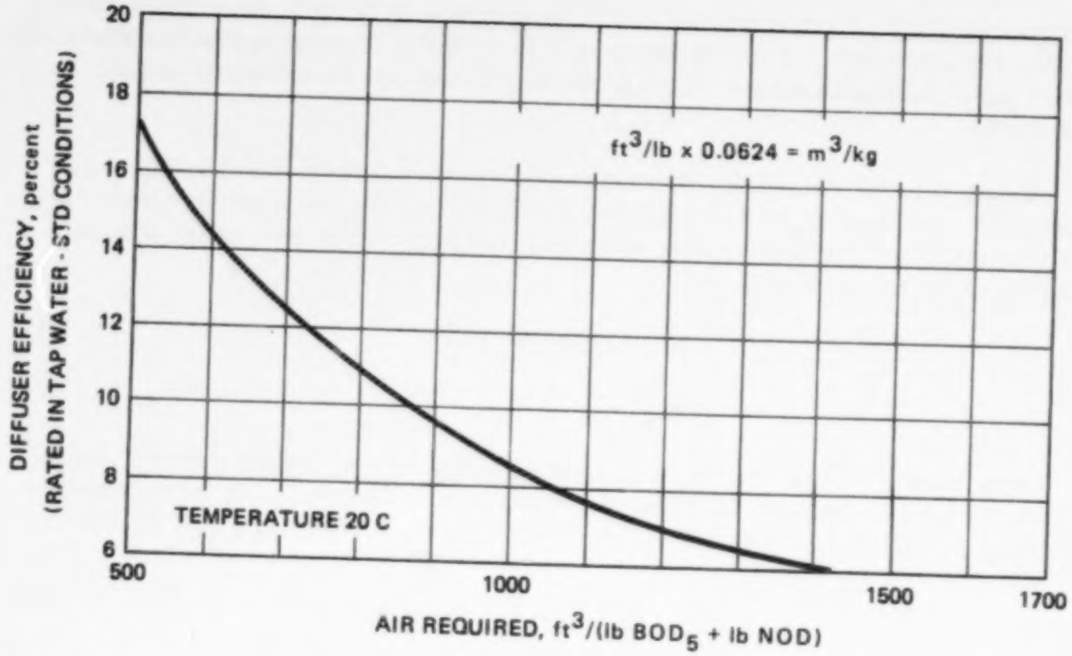

Figure 63. Air requirements for oxidation of carbonaceous and nitrogenous oxygen demand (Ref. 9).

In addition to determining the total air requirement, attention should also be given to air distribution within the aeration tanks. If the conventional (or plug flow) mode of operation is established as the normal operating procedure, the air requirements will be greatest at the head end of the aeration tanks.

Design Examples--

Air requirements for the coupled trickling filter-activated sludge process (or, alternatively, the two-stage trickling filter-activated sludge process with intermediate clarification) and the activated sludge process alone can be estimated. The coupled and two-stage processes will have (for preliminary estimating purposes) the same projected air supply requirements because it is assumed that only the settled BOD_5 (not including the sloughed solids) of the trickling filter effluent will exert significant oxygen demand in the aeration tank. Employing the flows and concentrations used in calculation of aeration tank size and assuming a BOD_5 peaking factor of 1.5 and a NOD peaking factor of 2.5, the estimated TOD from Equation 38 for either the coupled process or the two-stage process with intermediate clarification is:

$$\begin{aligned} \text{TOD} &= \text{BOD}_5 + \text{NOD} \\ &= 60 + 4.6\ (25 + 10) \\ &= 221\ \text{mg/l} \end{aligned} \tag{38}$$

For a 0.44-m^3/sec (10-mgd) flow, the average total oxygen-demand loading is equal to:

$$W = \frac{0.44\ (221)\ (86{,}400)}{1{,}000} = 8{,}400\ \text{kg/day}\ (18{,}520\ \text{lb/day})$$

For peak conditions:

$$W = \frac{0.44\ (86{,}400)}{1{,}000}\left[60\ (1.5) + 4.6\ (35)\ (2.5)\right] = 18{,}725\ \text{kg/day}\ (41{,}275\ \text{lb/day})$$

With a coarse bubble diffusion system a 6-percent transfer efficiency can be expected. From Figure 63, the unit air supply required is 90 m^3/(kg BOD_5 + kg NOD removed) (1,450 ft^3/lb). Assuming essentially complete removal of influent BOD_5 and NOD, the air requirement for average conditions is:

$$\frac{8{,}400\ (90)}{1{,}440} = 525\ m^3/\text{min}\ (18{,}540\ ft^3/\text{min})$$

For peak conditions, the air requirement is:

$$\frac{18{,}725\ (90)}{1{,}440} = 1{,}170\ m^3/\text{min}\ (41{,}325\ ft^3/\text{min})$$

For the activated sludge process alone:

$$\begin{aligned} \text{TOD} &= \text{BOD}_5 + \text{NOD} \\ &= 160 + 4.6\ (25 + 10) \\ &= 321\ \text{mg/l} \end{aligned} \tag{38}$$

For average conditions:

$$W = \frac{0.44\ (321)(86{,}400)}{1{,}000} = 12{,}200\ \text{kg/day}\ (26{,}900\ \text{lb/day})$$

For peak loads:

$$W = \frac{0.44\ (86{,}400)}{1{,}000}\left[160\ (1.5) + 4.6\ (35)\ (2.5)\right]$$

$$= 24{,}425\ \text{kg/day}\ (53{,}850\ \text{lb/day})$$

The air requirement for average conditions is:

$$\frac{12,200\ (90)}{1,440} = 765\ m^3/min\ (27,015\ ft^3/min)$$

For peak conditions, the air requirement is:

$$\frac{24,425\ (90)}{1,440} = 1,530\ m^3/min\ (54,025\ ft^3/min)$$

In comparison with the coupled trickling filter-activated sludge process or the two-stage process with intermediate clarification, projected air requirements for an activated sludge process alone are 46 percent and 31 percent higher for average and peak loads, respectively, under the assumed conditions of the design examples.

Physical Design

In addition to the process design information discussed above, important physical design considerations can be briefly outlined. These include reliability, flexibility, and use of existing structures. Each plant to be upgraded will be different, and specific actions taken will vary in all cases. Nevertheless, generalizations can be presented and specific examples discussed.

Reliability--

The increased stringency of discharge requirements will make treatment plant reliability an important design consideration in the future. Discharge of inadequately treated effluent, either on a long-term basis or as the result of short-term equipment breakdowns, will provoke increasingly frequent regulatory agency action against the discharger. Data presented in this report indicate that the coupled trickling filter-activated sludge process is inherently stable and can continuously produce an effluent low in BOD_5, suspended solids, ammonia nitrogen, and coliform organisms. Reliability in the face of equipment breakdowns is best accomplished by providing duplicate units with sufficiently low loading factors to permit shutdown of one process unit, or in some cases a unit process, without overloading others. Another means of incorporating reliability into design is to provide emergency storage which will allow bypassing of all or a portion of the plant flow for several days.

Both of these may be difficult when dealing with an existing plant. Provision of duplicate units requires both adequate space and an existing plant configuration amenable to such modifications. Emergency holding basins require large areas of land and may be economically most attractive where existing oxidation ponds can be used. In regard to the plants discussed in this report, the Livermore and Lompoc plants rely on emergency storage to provide reliability while the Corvallis plant utilizes duplicate units and unit process bypassing to ensure a high degree of treatment at all times.

Flexibility--

Flexibility has two principal aspects. The first is related to reliability and is manifested by the ease of bypassing individual plant components without significantly degrading effluent quality. As noted above, incorporation of such provisions may be hindered by the existing plant layout and can require considerable ingenuity on the part of the design engineer.

Flexibility also means the ability to accommodate future plant expansion and upgrading at reasonable cost and with minimum disruption of plant operations during construction. Future discharge requirements may necessitate nitrogen removal, phosphorus removal, filtration, and/or activated carbon sorption.

If nitrification is effected in a coupled trickling filter-activated sludge plant, nitrogen removal is best accomplished through biological denitrification, either with fixed-growth or suspended-growth reactors. Denitrification is the reduction of nitrate nitrogen to nitrogen gas, usually accomplished biologically under anoxic conditions. Because the effluent from a trickling filter-activated sludge plant will contain nitrogen in the nitrate form, denitrification can generally be easily and economically implemented.

Phosphorus removal is best accomplished by chemical precipitation, utilizing aluminum or iron salts or lime. The point of application may be the primary clarifiers, the secondary clarifiers, or a separate tertiary unit.

Because nitrification and addition of aluminum or iron compounds both result in the destruction of alkalinity, care must be taken when using the two in combination to avoid adverse effects from a drop in pH. If 10 mg/l of phosphorus are removed by alum addition ahead of the primary clarifier, about 70 mg/l of alkalinity will be destroyed and downstream chemical addition may be required to prevent low pH values in the nitrification process.

Lime addition to the primary clarifier may have an effect opposite from alum. The changes in alkalinity in this situation are dependent on lime dose (or desired pH) and raw wastewater quality. Cases of both increases and decreases in alkalinity with lime treatment may be found.

Primary effluent after lime treatment will typically have a pH between 9.5 and 11.0. This is higher than normally acceptable for discharge or introduction into downstream biological processes. To reduce the pH, the normal practice is to recarbonate the high pH primary effluent. Conventionally, this involves the introduction of gaseous carbon dioxide (CO_2) into the primary effluent in a reaction basin of at least 20 min detention time. Typically, carbon dioxide is either drawn from refrigerated storage, or furnace stack gases containing carbon dioxide are used. The recarbonation reaction can be thought of as the conversion of alkalinity in the hydroxide form (OH^-) to that in the bicarbonate form (HCO_3^-) as follows:

$$Ca(OH)_2 + 2\ CO_2 \rightarrow Ca(H_2CO_3)_2 \tag{43}$$

The important point to remember is that in combining nitrification with chemical addition for phosphorus removal, alkalinity and pH relationships must be carefully evaluated.

Rapid sand or multi-media filtration and carbon sorption columns are used to reduce residual suspended solids and organic compounds in treatment plant effluents. For most economical operation of these units, the influent concentrations of organics and suspended solids should be as low as possible. Because the coupled trickling filter-activated sludge process is capable of producing high quality effluents, it is very compatible with filtration and sorption.

Use of Existing Structures--

Possible incorporation of structures which would otherwise be abandoned into the flow diagram for the upgraded plant will necessarily have to be evaluated in each situation. The modification of the filter circulation sump at Livermore to accommodate the activated sludge process is probably a unique example which would not be duplicated elsewhere, but it does provide an excellent illustration of the way that existing structures can be used.

Generalizations may be made, however, about two types of facilities which will often be available for use in a coupled-process upgrading situation: the secondary clarifiers and the oxidation ponds which are often used to polish trickling filter effluent. Because trickling filter secondary clarifiers are usually designed on the basis of higher overflow rates than used for activated sludge secondary clarifiers and because they cannot accommodate the high MLSS levels and sludge removal rates required for the activated sludge process, additional secondary sedimentation capacity will normally be included in plant upgrading. It may be more economical to adapt the existing secondary clarifier to other uses than to modify it for use with activated sludge. At Livermore, it was converted to a primary sedimentation tank (as previously planned); at Corvallis, it was used as a chlorine contact tank; and at Lompoc, it was converted to a sludge thickener. The most appropriate use will be different at each plant.

Existing oxidation ponds may be converted to emergency holding basins as was done at Livermore and Lompoc. Another possibility is conversion to sludge lagoons. Often the entire existing pond area will not be required for these purposes. New feed and withdrawal lines will normally need to be constructed, and banks may need to be relocated.

Expected Effluent Quality

While plant performance will depend on the design criteria used at each installation as well as such factors as operator competence, it is worthwhile to project the maximum performance which can be expected from the coupled trickling filter-activated sludge process. With typical municipal wastewater, effluent BOD_5 and suspended solids concentrations of 10 mg/l can possibly be obtained, but 15 to 20 mg/l would be more conservative estimates. An effluent ammonia nitrogen concentration of 0.1 mg/l is expected from the Lompoc plant, and experience at Livermore indicates that 1.0 mg/l is not difficult to obtain. With high chlorine dosages and a chlorine contact tank which approximates plug flow, monthly median total coliform organism concentrations of 2.2 MPN/100 ml should be obtainable. For 7-day medians, 5.0 MPN/100 ml may be a more realistic value.

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APPENDIX A

CALIFORNIA REGIONAL WATER POLLUTION CONTROL BOARD NO. 2
SAN FRANCISCO BAY REGION

RESOLUTION NO. 239

PRESCRIBING REQUIREMENTS AS TO THE NATURE OF WASTE DISCHARGE INTO ARROYO LAS POSITAS NEAR THE SOUTHERLY PROLONGATION OF COLLIER CANYON ROAD AS PROPOSED BY THE CITY OF LIVERMORE

WHEREAS, the City of Livermore has filed with this Regional Water Pollution Control Board a report on proposed waste discharge, dated November 13, 1956, in accordance with Section 13054 of the Water Code of the State of California; and

WHEREAS, said report and subsequently submitted data provide the following information:

(a) The proposed waste discharge consists of mixed domestic sewage and industrial wastes;

(b) Change in manner of treatment and change in point of disposal are proposed;

(c) Two plant sites, designated as "Proposed Plant Site" and "Possible Alternate Plant Site", are proposed and their locations are shown on a map submitted as part of the report;

(d) Subsequent to the filing of the report on proposed waste discharge the City purchased the "Proposed Plant Site" on Arroyo Las Positas and advised the Board's staff that waste discharge requirements for the "Possible Alternate Plant Site" on Arroyo Mocho were no longer needed by the City;

(e) Treatment plant effluent from the proposed plant site will be discharged to Arroyo Las Positas at a point about 1000 feet downstream from the southerly prolongation of Collier Canyon Road;

(f) The present rate of flow is 0.70 mgd and the design rate of flow is 6.0 mgd;

(g) There are no industrial wastes being discharged at present and it is not known what type of industry will connect in the future;

(h) The City has asked that requirements be established for two conditions of discharge into Arroyo Las Positas as follows:

(1) Continuous discharge to the Arroyo after treatment in a plant consisting of the following: primary clarifier with separate sludge digestion, high rate trickling filter, secondary clarifier, chlorinator and chlorine contact tank, and oxidation ponds with a minimum of 30 days retention based on average dry weather flow;

(2) Intermittent discharge to the Arroyo after treatment in a plant consisting of the following: primary clarifier with separate sludge digestion; oxidation ponds with a minimum of 30 days retention based on average dry weather flow; and chlorinator and chlorine contact tank;

(i) Under condition (2) above, the City proposes:

(1) To dispose of all effluent from the oxidation ponds on land at its present sewage disposal site on Rincon Avenue or on land that is owned by, or under the control of, the City at the proposed plant site on Arroyo Las Positas whenever there is sufficient land available for disposal:

(2) To have sufficient land disposal area available at all times so that there will never be any waste discharge to the Arroyo unless the sewage flow at the treatment plant exceeds three times the average dry weather flow;

(3) To limit the rate of flow to the Arroyo to the actual flow at the sewage treatment plant minus three times the average dry weather sewage flow; and

WHEREAS, comments and recommendations relative to these proposals have been received and considered by the Board as follows:

(a) Memorandum from the State Department of Fish and Game, dated January 3, 1957;

(b) Memorandum from the State Department of Water Resources, dated February 7, 1957;

(c) Memorandum from the State Department of Public Health, dated February 7, 1957; and

WHEREAS, investigation by the staff of the Board discloses the following:

(a) The proposed plant site is at present well isolated from residential or commercial development; and the adjacent lands are used for agriculture;

(b) The land adjacent to Arroyo Las Positas below the proposed point of waste discharge is entirely agricultural at present except for the Santa Rita Rehabilitation Center Annex of the County of Alameda;

(c) There is a dairy, located on the Arroyo about 1.5 miles downstream from the proposed point of discharge, at which there are four residences within 200 feet of the Arroyo and at which milk cows pass through the Arroyo in going from the milking barn to the pastures;

(d) The waters of Arroyo de la Laguna are readily accessible to persons using the golf course at Hacienda Road;

(e) The waters of Alameda Creek in Niles Canyon are used for bathing, wading, fish propagation and fishing;

(f) The San Francisco Water Department has certain water rights for the surface diversion of water at the confluence of Laguna and Alameda creeks near Sunol;

(g) Alameda Creek recharges the southern Alameda county groundwater basin which is an important source of supply for domestic, agricultural, and industrial use;

THEREFORE, BE IT RESOLVED, that the waste of the City of Livermore shall be of the following quality for discharge to Arroyo Las Positas under conditions (1) and (2) as described above and that all samples shall be collected and analyzed in accordance with procedures given in the Tenth Edition of "Standard Methods for the Examination of Water, Sewage, and Industrial Wastes," by the American Public Health Association:

Condition 1 - Continuous Discharge

(a) The following requirements for the influent to the oxidation ponds are based on the proposal by the City that this influent will be chlorinated and that it will, at all times, be passed through properly designed operated and maintained oxidation ponds having a minimum retention time of 30 days based on average dry weather flow prior to discharge to the Arroyo;

(b) Any 24-hour conposite sample of the influent to the oxidation ponds shall not exceed the following limits:

5-day BOD - 55 ppm Maximum

Suspended Solids - 75 ppm Maximum

(c) The waste shall be adequately disinfected at all times and adequate disinfection, for the purpose of this requirement, is any process equivalent to the maintenance of 0.5 ppm of chlorine residual in the influent to the oxidation ponds after a chlorine contact period of 30 minutes;

Condition 2 - Intermittent Discharge

(a) The following requirements for the effluent from the oxidation ponds are based on the proposal by the City that this effluent will be chlorinated and that the effluent from the primary clarifier will, at all times, be passed through properly designed, operated and maintained oxidation ponds having a minimum retention time of 30 days, based on average dry weather flow, prior to chlorination and discharge to the Arroyo;

(b) The waste discharge to the Arroyo shall not contain more than 1 ml/liter/hour of settleable solids at any time;

(c) The waste discharged to the Arroyo shall not contain a most probable number of coliform organisms greater than 10 per milliliter at any time;

(d) The above coliform limit will be considered as being met if a chlorine residual of 2.0 ppm is maintained continuously after a chlorine contact time of 30 minutes;

BE IT FURTHER RESOLVED, that whenever there is any waste discharge by the City of Livermore to Arroyo Las Positas, the City shall make at least one determination per day of the chlorine residual at the end of the chlorine contact time and that these data together with a record of the corresponding sewage flow, at the time the chlorine residual is determined, shall be made available to this Board upon request;

BE IT FURTHER RESOLVED, that none of the following conditions shall occur in Arroyo Las Positas, Arroyo de la Laguna, or Alameda Creek as a result of any waste discharge by the City of Livermore:

Floating materials recognizable by eye as being of waste origin at any point;

Atmospheric odors recognizable as being of waste origin at any point;

Depletion of dissolved oxygen below 4.0 ppm in Arroyo de la Laguna or below 5.0 ppm in Alameda Creek;

Toxic concentrations of substances deleterious to humans, livestock, fish, or aquatic life;

BE IT FURTHER RESOLVED, that treatment or disposal of these wastes shall not cause atmospheric odors recognizable as being of waste origin at any point beyond the boundaries of the treatment or disposal sites;

BE IT FURTHER RESOLVED, that in the event that the City releases this waste to another person for use and/or disposal, it shall be necessary for the City to so advise this Board;

BE IT FURTHER RESOLVED, that it is the intention of this Board to formulate, at the earliest practicable date, a policy governing the mineral quality of all waste discharges in the Alameda Creek watershed and that the above requirements may at that time be modified to include limiting values of certain effluent constituents in addition to those listed above;

BE IT FURTHER RESOLVED, that in prescribing requirements for this discharge, it is the intent of this Board:

(a) To protect the surface and groundwaters, that may be affected by these proposed discharges, for domestic, agricultural and industrial purposes;

(b) To protect the waters of Alameda Creek for fish propagation, fishing, wading and bathing;

(c) To prevent nuisance and health hazards as a result of the proposed waste discharges;

BE IT FURTHER RESOLVED, that the Executive Officer is directed to transmit to the City of Livermore, copies of all the correspondence this Board has received and considered relative to these discharges, and that this Board requests said City to take note of the comments and recommendations contained in said correspondence;

BE IT FURTHER RESOLVED, that these requirements do not authorize the commission of any act causing injury to the property of another, nor protect the discharger from his liabilities under Federal, State, or local laws;

BE IT FURTHER RESOLVED, that if conditions should change materially at some time in the future it may be necessary to revise these requirements, and in such event this Board will take up the matter with the City of Livermore at that time;

BE IT FURTHER RESOLVED, that the requirements prescribed herein for continuous discharge to Arroyo Las Positas are based upon the present downstream development and that changes in the character or amount of downstream development along the Arroyo will be cause for this Board to review the adequacy of these requirements.

RALPH W. SHAFOR
Acting Chairman

March 21, 1957

I, John B. Harrison, hereby certify that the foregoing is a true and correct copy of Resolution No. 239, as adopted by the Regional Water Pollution Control Board of Region No. 2, at its regular meeting on March 21, 1957.

JOHN B. HARRISON
Executive Officer
Regional Water Pollution Control
Board No. 2

APPENDIX B

CALIFORNIA REGIONAL WATER POLLUTION CONTROL BOARD NO. 2
SAN FRANCISCO BAY REGION

RESOLUTION NO. 683

RESCINDING RESOLUTIONS NOS. 108 AND 239 AND THOSE PORTIONS OF RESOLUTION NO. 446 WHICH ARE DIRECTLY APPLICABLE ONLY TO THE CITY OF LIVERMORE, AND PRESCRIBING REQUIREMENTS AS TO THE NATURE OF WASTE DISCHARGE INTO ARROYO LAS POSITAS BY THE CITY OF LIVERMORE, ALAMEDA COUNTY.

I. WHEREAS, a Report on Waste Discharge, dated June 23, 1952, was filed, in accordance with Section 13054 of the State Water Code, with this Regional Water Pollution Control Board by the City of Livermore, hereinafter referred to as the discharger, on June 25, 1952, and requirements as to the nature of waste discharge were prescribed in this Regional Board's Resolution No. 108 adopted on August 21, 1952; and

II. WHEREAS, a Report on Waste Discharge, dated November 13, 1956, was filed with the Regional Board by the City of Livermore on November 14, 1956, and requirements as to the nature of waste discharge by the City into Arroyo Las Positas were prescribed in this Regional Board's Resolution No. 239 adopted on March 21, 1957; and

III. WHEREAS, at its February 21, 1963, regular meeting, this Regional Board adopted its Resolution No. 446 "PRESCRIBING ADDITIONAL REQUIREMENTS AS TO THE NATURE OF ALL WASTE DISCHARGES TO ALAMEDA CREEK OR ITS TRIBUTARIES ABOVE NILES, INCLUDING THE CITY OF LIVERMORE, VALLEY COMMUNITY SERVICES DISTRICT, CITY OF PLEASANTON, VETERANS ADMINISTRATION HOSPITAL AT LIVERMORE, CASTLEWOOD CORPORATION, CAMP PARKS, AND THE GENERAL ELECTRIC COMPANY'S VALLECITOS ATOMIC LABORATORY, ALAMEDA COUNTY"; and

IV. WHEREAS,
   A. The Livermore City Council, on November 18, 1963, adopted its Resolution No. 227-63 authorizing engineering studies and preparation of a report with recommendations for a program of sewage treatment and disposal to meet its present and future needs;

B. On April 22, 1964, the City's consultant submitted an engineering report which recommended certain treatment plant enlargements and improvements with continuous disposal of effluent into the Arroyo Las Positas;

C. At its September 17, 1964, regular meeting this Board was informed by the City that plans were underway to enlarge the treatment works to a capacity of 5.0 mgd (million gallons per day) with continuous disposal of effluent to the Arroyo Las Positas, as recommended in the aforementioned report;

D. On September 17, 1964, this Regional Board instructed its staff to prepare a report relative to the necessity for revision of the City's current requirements;

E. On October 21, 1964, the Executive Officer requested comments and recommendations from interested agencies and individuals as to the necessity for revision of the City's waste discharge requirements; and

V. WHEREAS, at its November 19, 1964, regular meeting this Board, after considering a staff report and other pertinent information relative to this discharge, directed its staff to review the existing requirements for the City of Livermore and to prepare a tentative Resolution containing revised requirements for this discharge; and

VI. WHEREAS, the aforesaid consultant's report and other data submitted by the discharger provide the following information:

A. The waste is sewage and industrial wastes;

B. The waste flow is estimated as follows:

1. Average dry weather flow for

August 1962	1.7 mgd (million gallons per day)
June, July and August 1963	1.8 mgd
September 1964	2.3 mgd

2. Design flow (existing plant) 2.5 mgd

3. Design flow (enlarged plant) 5.0 mgd;

C. The population served is estimated as follows:

1. Present - 23,000 persons

2. Design (existing plant) - approx. 29,000 persons

3. Design (enlarged plant) - approx. 58,000 persons;

D. Treated wastes will be disposed by discharge to Arroyo Las Positas; and

VII. WHEREAS, the Board has received and considered the following correspondence regarding this waste discharge:

A. Memorandum from State Department of Fish and Game, dated November 4, 1964;

B. Memoranda from State Department of Public Health, dated November 16, 1964, February 16, 1965, June 28, 1965, and July 13, 1965;

C. Memoranda from State Department of Water Resources, dated November 16, 1964, April 14, 1965, and July 12, 1965;

D. Letter from the Alameda County Board of Supervisors transmitting comments of:

1. The Alameda County Health Department (no date), and

2. The Alameda County Director of Public Works, dated November 6, 1964;

E. Letters from Mr. G. G. Jamieson, owner of Rancho Del Charro, dated October 27, 1964, and March 17, 1965;

F. Letter from Alameda County Planning Commission, dated October 28, 1964;

G. Letter from Mr. R. M. Wing, dated October 30, 1964;

H. Letter from Mr. Conrad Moldt, dated November 3, 1964;

I. Letter from Headquarters Camp Parks, dated 3 November 1964;

J. Letter from City of Pleasanton, dated November 6, 1964;

K. Letters from City of Livermore, dated November 12, 1964, March 12, 1965, May 20, 1965, May 26, 1965, June 25, 1965, and July 9, 1965;

L. Letters from Pleasanton Township County Water District, dated November 13, 1964, March 12, 1965, and May 18, 1965;

M. Letters from Alameda County Water District, dated November 13, 1964, January 14, 1965, March 15, 1965, July 15, 1965, and July 15, 1965;

N. Letter from City of Union City, dated November 19, 1964;

O. Letters from the Alameda County Health Department, dated March 9, 1965, and May 19, 1965;

P. Letter from the Alameda County Flood Control and Water Conservation District, Zone No. 7, dated May 18, 1965;

Q. Letters from the Pacific Water Conditioning Association and their consultants, dated May 10, 1965, May 20, 1965, and June 4, 1965;

R. Letter from the San Francisco Water Department, dated May 4, 1965;

S. Letter from Valley Community Services District, dated July 12, 1965 together with Resolution No. 23-65; and

VIII. WHEREAS, investigation by the staff of the Board discloses that:

A. Beneficial uses of the waters of Arroyo Las Positas, Arroyo de la Laguna and Alameda Creek downstream from the point of waste discharge include:

1. Recharge of groundwater basins of the Santa Rita subbasin (near El Charro Road) and the Niles Cone (near Niles), both basins being sources of supply for domestic, agricultural and industrial use;

2. Bathing;

3. Wading;

4. Fish propagation;

5. Fishing;

B. Land near the waste discharge point is used for agricultural purposes at present;

C. The City of Livermore proposes to develop a municipal airport and golf course adjacent to the waste discharge point;

D. The San Francisco Water Department has rights for the surface diversion of water at the confluence of Arroyo de la Laguna and Alameda Creek near Sunol and the Department also owns and uses the subsurface water rights to part of the Amador Valley south of Highway 50;

E. The City of Pleasanton owns and the Pleasanton Township County Water District uses the subsurface water rights to part of the Amador Valley south of Highway 50;

F. The waters of Arroyo de la Laguna are readily accessible to persons using the golf course at Hacienda Road;

G. The channels of Altamont Creek, Arroyo Las Positas, Arroyo de la Laguna and Alameda Creek are presently used, by the Alameda County Water District for the conveyance of water, imported from the Sacramento-San Joaquin Delta, to the Niles Cone for recharge of the groundwater basin;

H. Present plans call for ultimate discharge of South Bay Aqueduct water to the Alameda Creek system near Sunol and conveyance thence to Niles through the natural channels of said creek system; now

IX. THEREFORE, BE IT RESOLVED, that it is the intention of this Regional Water Pollution Control Board to:

A. Prevent nuisance, as defined in Water Code Section 13005, from being caused by this waste discharge or by treatment or conveyance of wastes tributary thereto;

B. Make or preserve the waters of Arroyo Las Positas and of Arroyo de la Laguna suitable for recharge of groundwaters that are a source of supply for domestic, agricultural and industrial use;

C. Make or preserve the waters of Alameda Creek suitable for the following beneficial uses:

1. Recharge of groundwaters that are a source of supply for domestic, agricultural and industrial use;
2. Bathing;
3. Wading;
4. Fish propagation;
5. Fishing;
6. Aesthetic enjoyment;

X. BE IT FURTHER RESOLVED, that in order to fulfill its intentions in the preceding paragraph, this Board prescribes the following requirements for this waste discharge:

A. FOR THE PREVENTION OF NUISANCE

1. The discharge to any ponds that may be used for treatment, shall not cause atmospheric odors recognizable as being of waste origin at any point outside the property of the discharger;
2. The discharge of Arroyo Las Positas or any tributary of Alameda Creek shall not cause:
   a. Floating, suspended, or deposited macroscopic particulate material of waste origin at any place;
   b. Atmospheric odors recognizable as being of waste origin at any place outside the property of the discharger;

c. Dissolved sulfide concentration of more than 0.1 mg/l at any place in the receiving water;

d. Foaming at any place;

e. Turbidity or discoloration in waters of the State at any place more than 100 feet from the point of discharge;

B. FOR THE PROTECTION OF BENEFICIAL WATER USES

1. The discharge to Arroyo Las Positas or any tributary of Alameda Creek shall not cause waters of the State to exceed the following limits of quality:

a. At any place in Arroyo Las Positas or Arroyo de la Laguna:

(1) Dissolved oxygen	4.0 mg/l, minimum
(2) Toxic concentrations of substances deleterious to humans or livestock	None;

b. At any place in Alameda Creek;

(1) Dissolved oxygen	5.0 mg/l, minimum
(2) Substances or combination of substances in concentration adverse to fish or aquatic life	None;

c. Any groundwater:

(1) Tastes or odors of waste origin	None
(2) Arsenic (As)	0.05 mg/l, maximum
(3) Barium (Ba)	1.0 mg/l, maximum
(4) Cadmium (Cd)	0.01 mg/l, maximum
(5) Chromium (hexavalent)	0.05 mg/l, maximum
(6) Cyanide (CN)	0.2 mg/l, maximum
(7) Fluoride (F)	1.0 mg/l, maximum
(8) Lead (Pb)	0.05 mg/l, maximum
(9) Selenium (Se)	0.01 mg/l, maximum
(10) Silver (Ag)	0.05 mg/l, maximum;

2. The quality of the waste discharged to Arroyo Las Positas or any tributary of Alameda Creek shall be maintained within the following limits:

a. For 6-hour composite samples collected one day each week throughout the year and made up of portions collected at hourly intervals in proportion to rate of flow at time of collection:

(1) 5-day BOD - No more than 50 percent of the samples in any one month to exceed 20 mg/l, and no sample to exceed 40 mg/l, maximum;
(2) Suspended solids - 20 mg/l, maximum
(3) Grease - 5 mg/l, maximum
(4) Settleable solids - 0.5 ml/l/hr, maximum;
(5) (a) Methylene Blue Active Substances or MBAS (as analyzed by the methylene blue method in accordance with the Eleventh Edition of "Standard Methods for the Examination of Water and Wastewater" by the American Public Health Association) - 1.0 mg/l, maximum

(b) The above requirement will be considered as being met whenever the discharger and all other waste discharges to Alameda Creek and its tributaries above Niles jointly develop and implement a program to monitor and control their discharges in such a manner that they can continuously demonstrate to the satisfaction of the Regional Board, that these discharges do not cause the MBAS concentration in the Arroyo de la Laguna at Verona Road to exceed 0.5 mg/l at any time, without allowing for the dilution effect of water imported by users downstream from Verona Road;

b. Any grab sample:

(1) Dissolved sulfide 0.1 mg/l, maximum
(2) Settleable solids 1.0 ml/l/hr, maximum;

C. FOR THE PREVENTION OF HAZARDS TO PUBLIC HEALTH

All waste discharged to the Arroyo Las Positas or to any tributary of Alameda Creek shall be adequately disinfected, and for the purpose of this Resolution, adequate disinfection shall mean any disinfection which produces a moving median coliform MPN value not exceeding 5.0 per 100 milliliters at some point in the treatment process, providing that moving median values shall be determined from:

1. Consecutive results obtained from samples collected on the previous five sampling days, and

2. Samples shall be collected at a frequency of at least five days each week, and

3. Samples shall be collected during those periods when the coliform MPN has been demonstrated to be at its daily maximum;

XI. BE IT FURTHER RESOLVED, that it is the intention of this Board, at the earliest practicable date to:

A. Adopt quality objectives for groundwaters in the Livermore groundwater basins;

B. Adopt a policy relative to making allowance for the dilution effect of water imported by users downstream from Verona Road;

C. Review the quality limits contained herein to assure their adequacy in achieving the objectives previously established by this Board in its Resolution No. 226, as amended by Paragraph XVI of Resolution No. 466, for groundwater in the Niles Cone;

XII. BE IT FURTHER RESOLVED, that this Board hereby adopts the following objectives for mineral quality of wastes discharged by the City of Livermore to the Arroyo Las Positas or to any tributary of Alameda Creek:

A. The monthly average concentration of constituents, as listed below and as determined by 6-hour composite samples collected weekly at the points designated, shall not exceed the sum of the increment listed for each plus its weighted average concentration in the water supply tributary to the discharge (as determined by monthly analyses of each water source):

	Increment as measured in -	
Constituent	The influent to the treatment facilities	The effluent from the treatment facilities
Total diss. solids	325 mg/l*	**
Chloride	75 mg/l*	**
Others	**	**

*To be reviewed by the Board whenever data becomes available which indicates to the staff of the Board and/or the City that review is necessary;
**To be established by the Board when sufficient data is available;

B. And, in furtherance of said objectives the discharger is required to file technical reports pursuant to Sections 13055 and 13055.1, California Water Code with this Regional Board as follows:

1. By September 1, 1965, a firm and detailed time schedule for actions which it proposes to take in order to regulate the mineral quality of wastes discharged into its sewerage system;

2. By January 1, May 1, and September 1, in each calendar year, a detailed report describing its progress in achieving such regulation;

All the aforementioned reports shall be transmitted with a motion of the City Council showing that that body has approved and is forwarding same to the Regional Board;

C. This Board will adopt specific requirements for mineral quality of wastes discharged by the City of Livermore to the Arroyo Las Positas or to any tributary of Alameda Creek, to replace the above objectives, whenever it is not satisfied that discharger is making reasonable progress toward meeting these objectives;

XIII. BE IT FURTHER RESOLVED, that in accordance with Sections 13055 and 13055.1 of the Water Code, the discharger is required to file technical reports on self-monitoring work performed according to detailed specifications developed pursuant to the Regional Board's Resolution No. 398, and that all sample collection and analysis for the purpose of determining compliance with the requirements prescribed in this Resolution shall be performed according to these detailed specifications;

XIV. BE IT FURTHER RESOLVED, that the Executive Officer is directed to transmit to the discharger copies of all correspondence this Board has received and considered relative to this waste discharge, and that the Board requests the discharger to take note of the comments and recommendations contained in said correspondence;

XV. BE IT FURTHER RESOLVED, that these requirements do not authorize the commission of any act causing injury to the property of another nor protect the discharger from his liabilities under Federal, State, or local laws;

XVI. BE IT FURTHER RESOLVED, that this Regional Water Pollution Control Board's Resolutions Nos. 108 and 239 and those portons of Resolution No. 446 which are directly applicable only to the City of Livermore are hereby rescinded;

XVII. BE IT FURTHER RESOLVED, that none of the requirements prescribed in this Resolution are a guarantee to the discharger of a capacity right in the receiving waters;

XVIII. BE IT FURTHER RESOLVED, that if conditions should change materially at some time in the future, it may be necessary to review these requirements, and in such event this Board will take up the matter with the responsible persons at this time;

XIX BE IT FURTHER RESOLVED, that pursuant to Section 13054, California Water Code this Regional Board requires the discharger to file a written report with the Board when the effluent flow, averaged for a calendar month during which there is no rainfall, exceeds 85% of the design capacity of the treatment facilities provided at that time; this report shall be accompanied by a duly adopted motion of the City Council approving said report and the report shall contain at least the following information:

A. Average flow for the month and date, time and volume of instantaneous peak flow during the month, as well as total flow for the calendar day during which said peak occurred;

B. Discharger's best estimate as to when the average dry weather effluent flow for the maximum month will reach design capacity;

C. Discharger's intentions with respect to actions to be taken and the timing of such actions, including engineering studies, etc., to expand existing facilities or to provide new or supplemental facilities before the waste flow volume reaches design capacity;

XX. BE IT FURTHER RESOLVED, that it is the intent of this Board to undertake investigations to determine the need for establishment of groundwater monitoring wells in the vicinity of the proposed waste discharges and the discharger will be required to establish and use such wells as a part of his monitoring program if such work is found by the Board to be necessary.

GRANT BURTON
Chairman

July 15, 1965

I, John B. Harrison, hereby certify that the foregoing is a true and correct copy of Resolution No. 683, as adopted by the Regional Water Pollution Control Board of region No. 2, at its regular meeting on July 15, 1965.

JOHN B. HARRISON, Executive Officer
REGIONAL WATER POLLUTION CONTROL BOARD NO. 2

APPENDIX C

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
SAN FRANCISCO BAY REGION

ORDER NO. 71-76

WASTE DISCHARGE REQUIREMENTS
FOR
THE CITY OF LIVERMORE

The California Regional Water Quality Control Board, San Francisco Bay Region finds that:

1. This Regional Board prescribed requirements for the City of Livermore, called the discharger below, as follows:

Resolution Number	Date Adopted	For Discharge To
683	July 15, 1965	Arroyo Las Positas
752	May 19, 1966	Land (Airport and Golf Course)
67-21	April 28, 1966	Four Temporary Septic Tanks
67-28	May 18, 1967	Land (Amends Resolution No. 752)
68-19	April 30, 1968	Arroyo Las Positas (Amends Resolution No. 683)
68-20	April 30, 1968	Time Schedule for Compliance with Mineral Quality Requirements
68-45	August 28, 1968	Amends Time Schedule of Resolution No. 67-21
68-50	August 28, 1968	Land (Amends Resolution No. 752 and 67-28)

2. Information in the Regional Board's files and in the City's report of waste discharge dated February 8, 1971,describes the existing waste discharge as approximately 3.6 mgd (million gallons per day) of sewage serving a population of 38,000. The current plant capacity is 5.0 mgd. Discharger proposes to increase plant capacity to 10.0 mgd and provide a higher degree of treatment. Effluent from the Livermore Water Reclamation Plant is used seasonally for agricultural irrigation and horticulture on the municipal airport and golf course grounds.

3. These wastes may affect the following beneficial water uses in Arroyo Las Positas, Arroyo de la Laguna and Alameda Creek:

   Recharge of groundwaters in the Santa Rita subbasin (near El Charro Road) and the Niles Cone (Near Niles) that are sources of supply for domestic, agricultural and industrial use;

   Bathing in Alameda Creek;

   Wading;

   Fish propagation in Arroyo de la Laguna and Alameda Creek;

   Fishing in Arroyo de la Laguna and Alameda Creek.

4. Land near the waste discharge point is used for residences, recreation, agriculture and commerce.

5. The San Francisco Water Department has rights for the surface diversion of water at the confluence of Arroyo de la Laguna and Alameda Creek near Sunol and the Department also owns and uses the subsurface water rights to part of the Amador Valley south of Highway 50. The Alameda County Water District has rights for surface water diversion to the Niles Cone groundwater basin from Alameda Creek between Mission Boulevard and Decoto Road for municipal, industrial and irrigation uses.

6. The City of Pleasanton owns and the Pleasanton Township County Water District uses the subsurface water rights to part of the Amador Valley south of Highway 50.

7. The waters of Arroyo de la Laguna are readily accessible to persons using the golf course at Hacienda Road.

8. The channel of Alameda Creek is and Arroyo de la Laguna may be used for the conveyance of water, imported from the Sacramento-San Joaquin Delta, to the Niles Cone for recharge of the groundwater basin.

9. A water quality management study for the Livermore Valley area is currently underway. The findings of this study will influence the nature of future treatment and disposal.

10. The Board adopted an Interim Water Quality Control Plan for the San Francisco Bay Basin on June 17, 1971.

11. The Board in a public meeting heard and considered all comments pertaining to this discharge.

IT IS HEREBY ORDERED, the discharger shall comply with the following:

A. Waste Discharge Requirements

1. The treatment or disposal of waste shall not create a nuisance as defined in Section 13050(m) of the California Water Code.

2. The discharge shall not cause:

   a. Floating, suspended, or deposited macroscopic particulate matter or foam, in waters of the State at any place;

   b. Bottom deposits or aquatic growths at any place;

   c. Alteration of temperature, turbidity, or apparent color beyond present natural background levels in waters of the State at any place;

   d. Visible, floating, suspended or deposited oil or other products of petroleum origin in waters of the State at any place;

   e. Waters of the State to exceed the following limits of quality at any point:

Dissolved oxygen in Arroyo Las Positas, Arroyo Mocho, or Arroyo de la Laguna	Minimum 5.0 mg/l Annual median 80% saturation.
Dissolved oxygen in Alameda Creek	Minimum 7.0 mg/l Annual median 90% saturation. When natural factors cause lesser concentrations, then this discharge shall not cause further reduction.
pH	No change in the natural ambient pH value by more than 0.1 pH unit.
Dissolved sulfide	0.1 mg/l maximum.
Other substances	No toxic or other substances shall be present in concentrations or quantities which will cause deleterious effects on aquatic biota, wildlife or waterfowl or which render any of these unfit for human consumption either at levels created in the receiving waters or as a result of biological concentration.

f. Groundwater at any point to contain tastes or odors of waste origin.

g. Groundwaters to exceed the following limits of quality at any point:

Arsenic (As)	0.05 mg/l, maximum
Barium (Ba)	1.0 mg/l, maximum
Cadmium (Cd)	0.01 mg/l, maximum
Chromium (hexavalent)	0.05 mg/l, maximum
Cyanide (Cn)	0.2 mg/l, maximum
Fluoride (F)	1.0 mg/l, maximum
Lead (Pb)	0.05 mg/l, maximum
Selenium (Se)	0.01 mg/l, maximum
Silver (Ag)	0.05 mg/l, maximum

3. The waste as discharged to waters of the State shall meet these quality limits at all times:

a. In any grab sample:

pH	7.0, minimum (6.5 when the natural ambient value is as low as 6.5) 8.5, maximum
Settleable matter	
The arithmetic average of any six or more samples collected on any day	0.5 ml/l/hr, maximum
80% of all individual samples collected during maximum daily flow over any 30-day period	0.4 ml/l/hr, maximum
Any sample	1.0 ml/l/hr, maximum
Dissolved sulfide	0.1 mg/l, maximum.

b. For 24-hour composite samples collected one day each week throughout the year and made up of portions collected at hourly intervals in proportion to rate of flow at time of collection:

Turbidity	10 units, maximum
5-day BOD and Suspended Solids	The waste as discharged, or at some point in the treatment process, shall meet quality

requirements equivalent to those which would result from conformance with Section 8047 of Title 17, California Administrative Code.

The monthly average concentration of constituents, as listed below, shall not exceed the sum of the increment listed for each, plus its weighted average concentration in the water supply tributary to the discharge (as determined by monthly analyses of each water source):

Constituent	Increment as measured in the influent to the treatment facilities
Total dissolved solids	325 mg/l*
Chloride	75 mg/l*
Others	**

*To be reviewed by the Board whenever data becomes available which indicates to the staff of the Board and/or the discharger that review is necessary.

**To be established by the Board when sufficient date are available.

Methylene Blue Active Substances or MBAS	1.0 mg/l, maximum.

c. The waste as discharged, or at some point in the treatment process, shall be adequately disinfected, and for the purpose of this Order adequate disinfection shall mean any disinfection which produces a moving median coliform MPN value not exceeding 2.2 per 100 milliliters at some point in the treatment process, providing that moving median values shall be determined from:

Consecutive results obtained from samples collected on the previous seven sampling days, and

Samples shall be collected at a frequency of at least seven days each week, and

Samples shall be collected during those periods when the coliform MPN has been demonstrated to be at its daily maximum.

4. Wastes used for irrigation shall comply with all requirements above, and:

   a. All equipment, including pumps, piping and valves, storage ponds, etc. which may at any time contain the waste shall be adequately and clearly identified with warning signs and discharger shall make all necessary provisions, in addition, to inform the public that the liquid contained therein is sewage and is unfit for human consumption.

   b. No wastes used for irrigation shall be allowed to escape from the property of discharger via surface flow.

5. This Regional Board hereby prohibits waste discharges by the City of Livermore to Arroyo Las Positas or any other tributary of Alameda Creek above Niles:

   a. In excess of 5.0 million gallons per day average dry weather flow for any calendar month until the City has provided facilities capable of meeting the requirements above.

   b. Subsequent to implementation of the program in "Provision 5" below, except in conformance with that program.

B. Provisions

1. This Order includes items numbered 1, 2, 6 and 7 of the attached "Reporting Requirements" dated August 28, 1970.

2. This Order includes items numbered 1, 2, 3, 4, 5 and 6 of the attached "Notifications", dated January 6, 1970.

3. The City of Livermore shall comply with the following time schedule and reporting program to assure compliance with the waste discharge requirements of this order except turbidity in receiving waters and effluent disinfection and suspended solids.

TASK	COMPLETION DATE	STATUS REPORT DATE
Demonstrate compliance with disinfection requirements and assure continuing compliance therewith at all times	December 1, 1971	December 15, 1971
Report on progress toward compliance with requirements for mineral increments		January 31, 1972

Complete Final Design	September 1, 1972	September 15, 1972
Authorization of Funding	October 1, 1972	October 15, 1972
Commence Construction	December 1, 1972	December 15, 1972
Report on progress toward compliance with requirements for mineral increments		January 31, 1973
Compliance with Requirements	September 1, 1973	September 15, 1973

The above compliance time schedule is subject to revision at the discretion of this Regional Board.

If the discharger fails to comply with the above time schedule, the Executive Officer is instructed to bring a recommendation on the initiation of formal enforcement proceedings to the Regional Board for its consideration.

4. It is the intention of this Board, at the earliest practicable date to:

   a. Adopt quality objectives for groundwaters in the Livermore groundwater basins.

   b. Review the quality limits contained herein to ensure their adequacy in achieving the objectives previously established by this Board in its Resolution No. 226, as amended by Paragraph XVI of Resolution No. 446, for groundwater in the Niles Cone.

5. This Regional Board adopts the following time schedule for long-term compliance with the objectives of Resolution No. 226, and to assure adequate protection of Livermore-Amador Valley groundwaters:

   The Valley Community Services District, the Cities of Livermore and Pleasanton and the Alameda County Flood Control and Water Conservation District - Zone 7, shall complete by April, 1972, a study which shall be the basis for a comprehensive water quality management program for the Alameda Creek System above Niles which is necessary to achieve the above and which is acceptable to this Board. The schedule, description, and program shall include consideration of at least the following items:

   Facilities necessary.

   Definition of the agency or agencies which will implement the program, operate facilities, etc.

   Means for securing a commitment by all agencies involved to implement the selected program.

In light of presently available information the agencies named above are required to complete necessary facilities and to implement this program no later than January 1975. An earlier date may be prescribed by this Regional Board after review of additional information.

6. This Board's Resolutions Nos. 683, 752, 67-21, 67-28, 68-19, 68-20, 68-45 and 68-50 are rescinded.

I, Fred H. Dierker, Executive Officer, do hereby certify the foregoing is a full, true, and correcto copy of an order adopted by the California Regional Water Quality Control Board, San Francisco Bay Region, on October 18, 1971.

Executive Officer

APPENDIX D

PERFORMANCE AND OPERATING PARAMETERS MEASURED AT THE LIVERMORE WATER RECLAMATION PLANT

The advanced nature of treatment and the stringency of the discharge requirements have resulted in unusually complete records of operation and performance being maintained at Livermore. Presented below are the parameters which are recorded on the monthly logs. Also listed are additional chemical analyses made for the City by an outside laboratory. Monthly averages for most of the parameters are tabulated in Appendix E.

Item	Frequency of Measurement[a]
1. Weather	
A. Temperature, F	
i. Maximum	D
ii. Minimum	D
B. Precipitation, in.	D
C. Evaporation, in.	D
D. Wind direction	D
2. Sewage Flow	
A. Influent	
i. Average	D
ii. Peak	D(P)
B. Effluent	
i. Average	D
ii. Maximum	D
iii. Minimum	D
iv. Airport use	D
v. Golf course use	D
vi. Farmland use	D
3. Grit Removed, ft^3	D

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[a] D = Daily, W = Weekly, I = Intermittently, P = Partial Record.

4. Chemical, Physical, and Biological Characteristics (for Sampling Techniques, see Table 11, Section 8.

Parameter	Raw Sewage[a]	Primary Effluent	Trickling Filter Effluent	Secondary Effluent[b]	Plant Effluent
BOD_5, mg/l	W	W	W	W (P)	W
SS, mg/l	W	W	W	W (P)	W
VSS, mg/l	W	W	W (P)[c]	W (P)	W
DO, mg/l				D (P)	D
Settl. Sol., ml/l	W	W	W (P)[c]	W (P)	W
Total Sol., mg/l	W	W	W (P)[c]		W
pH	W	W		W (P)	W
Grease, mg/l					W
MBAS, mg/l					W
NH_4^+ -N, mg/l		W	W	W (P)	W (P)
NO_2^- -N, mg/l				W (P)	W (P)
NO_3^- -N, mg/l				W (P)	W (P)
Turbidity, JTU					D (P)
Temperature, C					D (P)
TDS, mg/l	W		W (P)[c]		W
Fixed DS, mg/l	W		W (P)[c]		W
Spec. Cond., mm	W				W
Cl^-, mg/l	W				W

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[a] After 1970, raw sewage sample contains waste mixed liquor and lagoon supernatant recycled to headworks.

[b] For 1970 and later only.

[c] Nor recorded on monthly logs.

5. Additional analytical work done by Brown and Caldwell (Environmental Quality Analysts) on plant effluent.

   A. Monthly composite of weekly 24-hr composites:

      NO_2^- -N, NO_3^- -N, NH_4^+ -N, organic nitrogen, Cl^-, HCO_3^-, $CO_3^=$, $HPO_4^=$, $H_2PO_4^-$, Na^+ K^+, Ca^{++}, Mg^{++}, alkalinity, CO_2, hardness, dissolved residue, specific conductivity, pH, sodium percent, B (boron), F^- (fluoride), $PO_4^{\equiv}$, and turbidity.

   B. 6-mo or quarterly composite:

      Barium (Ba), Silver (Ag), Lead (Pb), Cadmium (Cd), Arsenic (As), Selenium (Se), Cyanide (CN) and Hexavalent Chromium (Cr).

6. Disinfection

A.	Lb chlorine for pre- and post-chlorination	D
B.	Time of sampling	D
C.	Flow rate, mgd	D
D.	Contact time, min	D
E.	Chlorine residual, mg/l	D
F.	pH, contact tank or effluent	D (P)
G.	Total coliform MPN's	
	i. For day	D
	ii. 5- or 7-day median	D

7. Mixed Liquor

A.	Air used, million ft^3/day	D
B.	DO, mg/l	D
C.	SS, mg/l, ml/l	D
D.	SVI	D
E.	Waste mixed liquor, 1,000 gpd	D

8. Sludge Handling and Treatment

A.	Secondary sludge pumped, mgd	D
B.	Return sludge, percent of influent flow	D
C.	Return sludge SS, mg/l	D
D.	Raw sludge, gpd	D
E.	Digested sludge to beds, gpd	D
F.	Digested sludge to lagoons, gpd	D
G.	Temperature of Digesters 1 and 2, F	D
H.	Gas produced by Digesters 1 and 2	D

9. Sludge Characteristics	
A. Total solids, percent	
i. Raw	I
ii. Digesters 1 and 2	I
iii. Digested	I
B. Volatile solids, percent	
i. Raw	I
ii. Digesters 1 and 2	I
iii. Digested	
C. Volatile acids, Digesters 1 and 2, mg/l	I
D. Alkalinity, Digesters 1 and 2, mg/l	I
E. pH, Digesters 1 and 2	I
10. Electric Power, KWH/day, and Natural Gas, 1,000 ft^3/day	D
11. Boiler Feed Water Hardness, mg/l; Specific Conductivity, micromhos; and Phosphate, mg/l	I

12. Equipment Operating Times, hours, monthly

A. Well pump
B. Drain pump (2)
C. Barminutor
D. Comminutor
E. Preaeration blower (2)
F. Primary sludge collector (2)
G. Scum collector (2)
H. Sludge airlift (2)
I. Grit washer
J. Air compressor (2)
K. No. 3 HP water pump (2)
L. No. 3 LP water pump (2)
M. Raw sewage pump (2)
N. Digester gas circulation pump (2)
O. Sludge circulation pump (2)
P. Raw sludge pump (2)
Q. Filter circulation pump (3)
R. Activated sludge influent pump (2)
S. Aeration blower (3)
T. RAS pump (2)
U. Secondary sludge collection
V. Effluent irrigation pump (2)

APPENDIX E

MONTHLY OPERATING AND PERFORMANCE DATA

Presented in this appendix are tabulations of monthly averages of operating and performance data for the years 1968 through 1974. Included are most of the items listed in Appendix D. The data are presented in seven table formats designated Table E-1, Table E-2, etc. Seven tables are used for each format, and each table represents one year. Thus, for example, Table E-1A represents 1968, and Table E-1G represents 1974.

The material in each table format is summarized below:

E-1	Wastewater Flows, Influent Characteristics, and Weather Characteristics
E-2	Preliminary Treatment, Primary Treatment, and Trickling Filter Treatment
E-3	Activated Sludge Aeration Tank and Secondary Treatment
E-4	Disinfection, Final Effluent Characteristics, and Overall Plant Performance
E-5	Solids Handling and Treatment
E-6	Power, Chemicals, and Labor
E-7	Final Effluent Heavy Metals.

TABLE E-1A. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1968

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	2.7[f]	2.3[f]	2.4[f]	2.4[f]	3.0	3.3	3.5	3.7	3.9	4.0	4.3	3.9	4.3	2.3[f]	3.7[g]
Daily peak, mgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Reclamation Use, Mil Gal[b]															
Airport	0	0	0	0	1.0	4.3	3.9	5.4	0	0	0	0	5.4	0	14.6[e]
Golf course	0	0	1.2	9.1	11.6	18.0	15.2	13.9	11.1	5.2	0.9	1.1	18.0	0	87.3[e]
Farmland	-	-	-	-	5.9	13.8	11.7	16.3	19.9	0	0	0	19.9	0	67.6[e]
Total	0	0	1.2	9.1	18.5	36.1	30.8	35.6	31.0	5.2	0.9	1.1	36.1	0	169.5[e]
Influent Characteristics															
BOD_5, mg/l	170	160	210	190	180	180	210	200	200	180	160	160	210	160	180
BOD_5, lb/day[c]	3,800	3,100	4,200	3,800	4,800	5,000	6,100	6,200	6,500	6,000	5,700	5,200	6,500	3,100	5,000
Suspended solids, mg/l	170	180	300	260	230	230	240	280	160	150	170	160	300	150	210
Suspended solids, lb/day[c]	3,800	3,500	6,000	5,200	6,100	6,400	7,000	8,700	5,200	5,000	6,000	5,200	8,700	3,500	5,700
Volatile suspended solids, mg/l	150	160	230	220	200	190	200	210	120	130	140	140	230	120	170
Settleable solids, ml/l	9.0	7.3	12.5	11.0	7.3	7.0	11.5	10.6	9.0	8.5	7.1	6.9	12.5	6.9	8.9
pH	7.6	7.6	7.5	7.5	7.6	7.6	7.6	7.5	7.4	7.4	7.4	7.4	7.6	7.9	7.5
Total solids, mg/l	1,110	1,120	1,480	1,480	1,400	1,300	1,240	1,280	1,150	930	860	930	1,480	860	1,180
Total dissolved solids, mg/l	840	940	1,180	1,220	1,170	1,080	1,000	1,010	990	700	710	780	1,220	700	970
Fixed dissolved solids, mg/l	610	780	970	950	1,000	900	730	800	820	540	530	610	1,000	530	770
Specific conductivity, micromhos	1,300	1,530	1,680	2,040	1,890	1,930	1,660	1,730	1,550	1,200	1,120	1,150	2,040	1,120	1,560
Chlorides, mg/l	300	340	440	430	450	420	380	380	400	210	200	230	450	200	350
Weather Characteristics															
Air temperature, F															
Daily maximum	56	63	67	73	74	85	87	84	84	-	59	55	87	55	72
Daily minimum	31	44	41	39	45	52	54	54	51	-	43	34	54	31	44
Precipitation, in.[d]	4.18	0.91	2.45	0.58	0.11	0	0	0.05	0	0.10	2.27	3.08	4.18	0	13.73[e]

[a] mgd x 0.044 = m³/sec

[b] mil gal x 3,800 = m³

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Estimate

[g] Excludes estimates

TABLE E-1B. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1969

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	3.9	4.9	4.5	4.1	3.1	3.5	-	-	-	3.7[f]	-	3.1	4.9	3.1	4.0[g]
Daily peak, mgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Reclamation Use, Mil Gal[b]															
Airport	0	0	0.2	0	0	3.4	12.9	8.0	4.4	0.3	6	0	12.9	0	29.2[e]
Golf course	0	0	1.9	6.6	14.1	14.0	15.9	22.7	13.1	7.8	3.2	1.2	22.7	0	100.5[e]
Farmland	0	0	0	0	0.5	15.2	17.8	20.4	14.0	0	0	0	20.4	0	67.9[e]
Total	0	0	2.1	6.6	14.6	32.6	46.6	51.1	31.5	8.1	3.2	1.2	51.1	0	197.6[e]
Influent Characteristics															
BOD_5, mg/l	150	160	200	190	140	200	210	220	170	180	200	190	220	140	180
BOD_5, lb/day[c]	4,900	6,500	7,500	6,800	4,700	5,800	-	-	-	5,600	-	6,300	7,500	4,700	6,000
Suspended solids, mg/l	200	280	180	180	140	130	160	220	160	170	200	180	280	130	180
Suspended solids, lb/day[c]	6,500	11,400	6,700	6,400	4,700	3,800	-	-	-	5,300	-	6,000	11,400	3,800	6,400
Volatile suspended solids, mg/l	160	200	150	150	120	70	140	190	150	150	170	150	200	70	150
Settleable solids, ml/l	9.0	8.4	8.7	9.0	7.1	7.3	9.1	8.9	7.8	9.6	9.5	8.8	9.6	7.1	8.6
pH	7.6	7.5	7.6	7.5	7.5	7.5	7.4	7.3	7.5	7.4	7.4	7.3	7.6	7.3	7.5
Total solids, mg/l	920	960	1,020	940	890	850	920	880	860	880	870	860	1,020	850	900
Total dissolved solids, mg/l	720	640	810	840	760	720	750	710	700	710	660	680	840	640	730
Fixed dissolved solids, mg/l	550	400	590	540	460	510	570	530	440	420	480	450	590	400	500
Specific conductivity, micromhos	1,080	1,020	1,050	1,350	1,290	1,040	1,160	1,100	1,070	1,140	1,080	1,020	1,350	1,020	1,120
Chlorides, mg/l	210	150	230	230	200	210	190	140	140	170	140	150	230	140	180
Weather Characteristics															
Air temperature, F															
Daily maximum	54	56	64	66	78	74	90	94	88	74	69	62	94	54	72
Daily minimum	36	38	37	41	48	54	54	53	53	45	37	40	54	36	45
Precipitation, in.[d]	5.64	5.25	0.66	1.23	0.05	0.01	0	0	0.03	0.99	0.46	2.24	5.64	0	16.56[e]

[a] mgd x 0.044 = m^3/sec

[b] mil gal x 3,800 = m^3

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Estimate

[g] Excludes estimate

TABLE E-1C. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1970

Parameter	Month														
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	High	Low	Average
Influent Flow															
Average, mgd[a]	3.8	3.3	3.5	3.9	3.8	3.4	4.0	4.1	3.7	3.8	3.9	3.8	4.1	3.3	3.8
Daily peak, mgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Reclamation Use, Mil Gal[b]															
Airport	- 0	0	0	0	4.5	6.6	8.7	5.4	13.9	0	0	0	13.9	0	39.1[e]
Golf course	0.4	1.1	6.0	11.3	16.3	16.5	11.7	18.4	15.6	7.9	2.5	0	18.4	0	107.7[e]
Farmland	0	0	0	0.1	7.9	16.9	24.2	31.9	27.6	7.9	0	0	31.9	0	116.5[e]
Total	0.4	1.1	6.0	11.4	28.7	40.0	44.6	55.7	57.1	15.8	2.5	0	57.1	0	263.3[e]
Influent Characteristics[f]															
BOD_5, mg/l	230	240	250	250	240	220	220	260	200	250	260	250	260	200	240
BOD_5, lb/day[c]	7,300	6,600	7,300	8,100	7,600	7,300	7,300	8,900	6,200	7,900	8,500	7,900	8,900	6,200	7,600
Suspended solids, mg/l	290	300	220	230	230	200	220	210	220	210	270	210	300	200	230
Suspended solids, lb/day[c]	9,200	8,200	6,400	7,500	7,300	6,600	7,300	7,200	6,800	6,600	8,000	6,600	9,200	6,400	7,400
Volatile suspended solids, mg/l	200	280	180	170	200	150	190	180	200	180	230	160	280	150	200
Settleable solids, ml/l	11.2	10.5	13.0	10.7	11.7	10.9	11.0	14.2	12.3	10.8	12.6	11.5	14.2	10.5	11.7
pH	7.4	7.3	7.7	7.4	7.3	7.5	7.4	7.1	7.4	7.5	7.3	7.2	7.7	7.1	7.4
Total solids, mg/l	940	1,000	980	960	910	910	910	930	910	900	930	1,010	1,010	900	940
Total dissolved solids, mg/l	650	700	760	730	680	710	690	710	680	690	670	800	800	650	710
Fixed dissolved solids, mg/l	360	490	570	470	530	470	530	490	420	500	470	550	570	360	490
Specific conductivity, micromhos	1,040	1,230	1,120	1,150	1,170	1,060	1,220	1,080	1,070	1,030	950	1,150	1,230	950	1,100
Chlorides, mg/l	150	180	170	180	160	150	160	160	170	150	130	160	180	130	160
Weather Characteristics															
Air temperature, F															
Daily maximum	60	64	68	68	79	80	88	86	87	74	66	56	88	56	73
Daily minimum	41	39	40	37	48	51	55	50	49	44	44	37	55	37	44
Precipitation, in.[d]	5.79	1.59	0.90	0.41	0.70	0.28	0	0	0	0.38	5.52	4.76	5.79	0	20.33[e]

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[a] mgd x 0.044 = m^3/sec

[b] mil gal x 3,800 = m^3

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Includes waste mixed liquor and lagoon supernatant for 1970 and later.

TABLE E-1D. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1971

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	3.8	3.8	3.7	3.7	3.9	4.0	3.8	3.9	4.1	4.0	3.9	3.8	4.1	3.7	3.9
Daily peak, mgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Reclamation Use, Mil Gal[b]															
Airport	0	0	0.4	3.2	0	6.2	1.4	6.7	7.7	0	0	0	7.7	0	25.6[e]
Golf course	0.5	-	4.5	8.1	13.5	16.6	21.6	20.4	15.2	8.2	4.8	0.4	21.6	0.5	113.8[e]
Farmland	0	0	0	3.9	0	10.4	32.4	33.0	28.8	13.4	0	0	33.0	0	129.9[e]
Total	0.5	-	4.9	15.2	13.5	41.2	55.4	60.1	51.7	21.6	4.8	0.4	60.1	0.5	269.3[e]
Influent Characteristics[f]															
BOD_5, mg/l	240	220	240	240	230	190	200	170	180	190	220	240	240	170	210
BOD_5, lb/day[c]	7,600	7,000	7,400	7,400	7,500	6,300	6,300	5,500	6,200	6,300	7,200	7,600	7,600	5,500	7,000
Suspended solids, mg/l	250	200	220	240	240	260	220	260	190	240	210	240	260	190	230
Suspended solids, lb/day[c]	7,900	6,400	6,800	7,400	7,800	8,600	6,900	8,400	6,500	8,000	6,900	7,600	8,600	6,400	7,400
Volatile suspended solids, mg/l	210	180	180	200	180	210	180	220	140	210	170	200	220	140	190
Settleable solids, ml/l	11.8	11.1	11.4	11.6	11.1	16.1	12.5	14.3	9.5	16.9	13.6	13.6	16.9	9.5	12.8
pH	6.9	7.4	7.3	7.3	7.3	7.2	7.4	6.8	6.9	6.8	7.3	7.3	7.4	6.8	7.2
Total solids, mg/l	1,020	1,020	1,020	940	1,010	940	920	900	940	870	870	920	1,020	870	950
Total dissolved solids, mg/l	800	810	800	700	770	690	710	670	800	630	670	690	810	630	730
Fixed dissolved solids, mg/l	590	580	620	480	540	500	540	450	450	440	460	490	620	440	510
Specific conductivity, micromhos	1,210	1,210	1,240	1,120	1,030	1,160	1,080	1,000	1,160	920	1,030	1,070	1,240	920	1,100
Chlorides, mg/l	170	160	170	150	160	140	160	130	110	110	120	130	170	110	140
Weather Characteristics															
Air temperature, F															
Daily maximum	59	61	64	67	71	79	87	87	85	75	65	55	87	55	71
Daily minimum	37	35	39	40	45	50	53	55	52	41	36	38	55	35	43
Precipitation, in.[d]	1.30	0.32	1.76	0.97	0.44	0	0	0.05	0.07	0.04	0.56	3.79	3.79	0	9.30[e]

[a] mgd x 0.044 = m^3/sec

[b] mil gal x 3,800 = m^3

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Includes waste mixed liquor and lagoon supernatant for 1970 and later.

TABLE E-1E. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1972

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	3.9	4.1	4.4	4.5	4.6	4.6	4.8	4.9	4.8	4.0	5.1	4.9	5.1	3.9	4.6
Daily peak, mgd	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Reclamation Use, Mil Gal[b]															
Airport	0	0	1.9	5.3	10.4	12.6	4.2	15.3	11.8	0	0	0	15.3	0	61.5[e]
Golf course	0	0	9.2	15.2	21.5	21.8	23.2	19.9	10.9	2.9	0	0	23.2	0	124.6[e]
Farmland	0	0	0	10.2	19.4	11.0	22.8	19.7	13.1	4.8	0	0	22.8	0	101.0[e]
Total	0	0	11.1	30.7	51.3	45.4	50.2	54.9	35.8	7.7	0	0	54.9	0	287.1[e]
Influent Characteristics[f]															
BOD_5, mg/l	230	240	240	300	180	280	220	240	230	200	200	210	300	180	230
BOD_5, lb/day[c]	7,500	8,200	8,800	11,000	6,800	10,700	8,400	9,800	9,200	6,700	8,500	8,600	11,000	6,700	8,700
Suspended solids, mg/l	340	260	290	230	210	240	280	290	250	250	280	250	340	210	260
Suspended solids, lb/day[c]	11,100	8,900	10,600	8,400	7,900	9,200	10,700	11,800	10,000	8,400	11,900	10,200	11,900	7,900	9,900
Volatile suspended solids, mg/l	220	210	230	190	120	160	60	250	180	190	230	220	250	60	190
Settleable solids, ml/l	14.8	14.0	9.7	15.1	12.5	11.2	17.0	15.8	13.4	14.0	15.9	13.9	17.0	9.7	13.9
pH	7.4	7.3	7.1	7.3	7.1	7.6	7.3	7.2	7.4	7.2	7.3	7.2	7.6	7.1	7.3
Total solids, mg/l	1,070	870	900	910	880	920	1,060	1,000	890	790	940	1,080	1,080	790	940
Total dissolved solids, mg/l	730	610	610	680	700	680	720	710	640	540	650	830	830	540	680
Fixed dissolved solids, mg/l	470	440	410	450	390	450	630	520	460	280	500	580	630	280	460
Specific conductivity, micromhos	970	1,080	1,070	1,100	980	990	960	950	1,060	1,030	1,040	1,110	1,110	950	1,030
Chlorides, mg/l	140	140	130	120	120	120	140	200	150	130	140	180	200	120	140
Weather Characteristics															
Air temperature, F															
Daily maximum	55	62	71	71	78	85	89	88	81	71	61	51	89	51	72
Daily minimum	31	40	42	41	48	51	56	55	50	50	39	32	56	31	45
Precipitation, in.[d]	0.75	0.84	0.16	0.54	0	0.06	0	0	0.46	2.59	5.06	2.17	5.06	0	12.63[e]

[a] mgd x 0.044 = m^3/sec

[b] mil gal x 3,800 = m^3

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Includes waste mixed liquor and lagoon supernatant for 1970 and later.

TABLE E-1F. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1973

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	5.3	5.4	5.5	3.0	2.9	3.1	3.8	3.8	3.8	3.8	4.2	4.1	5.5	2.9	4.1
Daily peak, mgd	-	-	-	-	-	-	-	-	5.3	5.2	6.0	6.2	6.2	5.2	5.7
Reclamation Use, Mil Gal[b]															
Airport	0	0	0	0	17.2	0	7.7	5.0	5.6	0	0	0	17.2	0	35.5[e]
Golf course	0	0	0.8	10.8	17.4	21.2	16.4	-	-	-	1.1	0	21.2	0	67.7[e]
Farmland	0	0	0	0	0	0	10.0	20.3	10.3	0.9	0	0	20.3	0	41.5[e]
Total	0	0	0.8	10.8	34.6	21.2	34.1	25.3	15.9	0.9	1.1	0	34.6	0	144.7[e]
Influent Characteristics[f]															
BOD_5, mg/l	210	220	240	240	220	210	210	220	190	210	220	230	240	190	220
BOD_5, lb/day[c]	9,300	9,900	11,000	6,000	5,300	5,400	6,700	7,000	6,000	6,700	7,700	7,900	11,000	5,300	7,200
Suspended solids, mg/l	210	230	230	280	240	270	230	260	270	180	250	230	280	180	240
Suspended solids, lb/day[c]	9,300	10,400	10,500	7,000	5,800	6,900	7,300	8,300	8,500	5,700	8,800	7,900	10,500	5,700	8,000
Volatile suspended solids, mg/l	180	160	190	180	190	220	190	200	230	140	220	170	230	140	190
Settleable solids, ml/l	11.1	13.6	13.8	13.4	16.2	14.6	14.3	13.9	14.8	10.9	13.6	13.9	16.2	10.9	13.7
pH	7.5	7.3	7.4	7.2	7.2	7.2	7.2	7.3	7.2	7.3	7.2	7.2	7.5	7.2	7.3
Total solids, mg/l	1,100	1,120	1,160	1,110	1,040	1,020	980	1,020	960	920	990	970	1,160	920	1,030
Total dissolved solids, mg/l	880	890	930	830	800	750	750	760	690	740	730	740	930	690	790
Fixed dissolved solids, mg/l	520	530	700	660	600	550	570	510	520	380	480	530	700	380	550
Specific conductivity, micromhos	1,300	1,280	1,380	1,400	1,316	1,230	1,100	1,260	1,050	1,190	1,100	1,050	1,400	1,050	1,220
Chlorides, mg/l	200	220	230	220	190	170	200	210	170	150	180	180	230	150	190
Weather Characteristics															
Air temperature, F															
Daily maximum	55	-	-	72	79	87	86	85	81	76	62	54	87	54	74
Daily minimum	34	-	-	42	48	53	54	54	52	48	43	32	54	32	46
Precipitation, in.[d]	5.05	4.26	2.54	0.44	0.04	0	0	0	0.07	1.75	5.06	2.74	5.06	0	21.95[e]

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[a] mgd x 0.044 = m^3/sec

[b] mil gal x 3,800 = m^3

[c] lb/day x 0.45 = kg/day

[d] in. x 2.54 = cm

[e] Total

[f] Includes waste mixed liquor and lagoon supernatant for 1970 and later.

TABLE E-1G. WASTEWATER FLOWS, INFLUENT CHARACTERISTICS, AND WEATHER CHARACTERISTICS, 1974

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Influent Flow															
Average, mgd[a]	4.0	3.9	4.0	4.2	4.0	3.5	3.3	4.3	4.5	4.6	4.6	4.6	4.6	3.3	4.1
Daily peak, mgd	5.8	6.1	5.8	6.3	6.4	6.4	6.4	6.6	6.9	6.5	6.8	-	6.9	5.8	6.3
Reclamation Use, Mil Gal[b]															
Airport	0	0	0	0	5.2	0	6.2	13.5	0	0.5	0.2	-	13.5	0	25.6[e]
Golf course	0	0	0.3	2.8	13.4	11.9	10.8	9.3	5.2	5.1	0.4	-	13.4	0	59.2[e]
Farmland	0	0	0	0	10.4	13.3	8.9	15.1	2.4	0	0	-	15.1	0	50.1[e]
Total	0	0	0.3	2.8	29.0	25.2	25.9	37.9	7.6	5.6	0.6	-	37.9	0	134.9[e]
Influent Characteristics[f]															
BOD_5, mg/l	220	230	250	200	240	220	200	220	200	190	170	-	250	170	210
BOD_5, lb/day[c]	7,300	7,500	8,300	7,000	8,000	6,400	5,500	7,900	7,500	7,300	6,500	-	8,300	5,500	7,200
Suspended solids, mg/l	300	230	230	300	200	220	240	280	220	220	280	-	300	200	250
Suspended solids, lb/day[c]	10,000	7,500	7,700	10,500	6,700	6,400	6,600	10,000	8,200	8,400	10,700	-	10,700	6,400	8,400
Volatile suspended solids, mg/l	260	200	190	240	160	180	200	240	180	180	250	-	260	160	210
Settleable solids, ml/l	19.5	10.1	15.0	15.8	11.8	13.3	13.2	14.6	12.9	15.4	14.5	-	19.5	11.8	14.9
pH	7.2	7.5	7.4	7.1	7.4	7.1	7.1	7.1	7.0	7.0	7.1	-	7.5	7.0	7.2
Total solids, mg/l	1,020	1,000	1,060	1,060	970	880	910	940	870	820	990	-	1,060	820	960
Total dissolved solids, mg/l	720	780	830	750	700	650	660	670	650	600	530	-	830	530	700
Fixed dissolved solids, mg/l	580	600	620	570	560	520	510	530	510	400	540	-	620	400	540
Specific conductivity, micromhos	1,200	1,170	1,160	1,260	1,110	1,100	1,010	1,070	1,090	1,100	1,160	-	1,260	1,010	1,130
Chlorides, mg/l	180	180	190	180	190	150	160	150	140	130	140	-	190	130	160
Weather Characteristics															
Air temperature, F															
Daily maximum	56	59	63	67	73	81	86	88	88	78	63	-	88	56	73
Daily minimum	37	35	43	42	46	52	54	54	51	47	40	-	54	35	45
Precipitation, in.[d]	1.41	0.87	2.26	1.43	0	0	0.17	0	0	0.82	0.56	1.31	2.26	0	8.83[e]

[a] mgd × 0.044 = m^3/sec

[b] mil gal × 3,800 = m^3

[c] lb/day × 0.45 = kg/day

[d] in. × 2.54 = cm

[e] Total

[f] Includes waste mixed liquor and lagoon supernatant for 1970 and later.

TABLE E-2A. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1968

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft³/day*	1.8	1.7	2.2	2.3	2.6	3.8	3.8	3.6	3.6	2.7	1.3	1.4	3.8	1.3	2.5
Chlorine added, mg/l	-	-	-	-	1.6	1.7	1.5	1.4	1.3	1.3	1.2	-	1.7	1.2	1.4
Primary Treatment Effluent Characteristics															
BOD_5, mg/l	120	120	130	140	120	100	150	150	180	180	150	160	180	100	140
Suspended solids, mg/l	63	72	97	95	93	68	95	120	160	75	110	91	160	63	95
Volatile suspended solids, mg/l	56	69	49	81	81	50	75	94	130	64	100	84	130	49	78
Settleable solids, ml/l	0.1	<0.1	0.1	0.4	0.1	0.1	0.6	1.6	2.6	<0.1	0.1	3.3	3.3	<0.1	0.7
Total solids, mg/l	950	1,030	1,260	1,300	1,400	1,300	1,110	1,110	1,070	780	830	830	1,400	780	1,080
Ammonia nitrogen, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
pH	7.6	7.6	7.6	7.5	7.6	7.6	7.6	7.5	7.4	7.4	7.4	7.5	7.6	7.4	7.5
Primary Treatment Removals, Percent															
BOD_5	29	25	38	26	33	44	29	25	10	17	6	0	44	0	22
Suspended solids	63	60	68	63	60	83	60	57	0	50	35	43	83	0	55
Trickling Filter Effluent Characteristics															
BOD_5, mg/l	88	76	70	93	79	61	97	109	110	140	110	110	140	61	94
Suspended solids, mg/l	100	110	92	120	97	97	100	120	190	140	160	110	190	92	120
Volatile suspended solids, mg/l	95	110	39	89	87	82	84	90	130	120	160	100	160	39	100
Ammonia nitrogen, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Trickling Filter Effluent Changes, Percent															
BOD_5 reduction	27	37	46	34	34	39	35	33	39	7	27	31	46	7	33
Suspended solids increase	59	53	-5	26	4	43	5	0	19	87	45	21	87	-5	26
Volatile suspended solids increase	70	59	-20	10	7	41	12	-4	0	88	60	19	88	-20	28

*ft³/day x 0.028 = m³/day

TABLE E-2B. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1969

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft^3/day[a]	1.0	0.8	1.2	1.6	1.2	1.5	1.3	1.9	1.4	1.4	1.6	1.3	1.9	0.8	1.4
Chlorine added, mg/l	1.1	1.0	1.2	0	0	1.8	-	-	-	2.4	-	2.5	2.5	0	1.7[b]
Primary Treatment Effluent Characteristics															
BOD$_5$, mg/l	140	110	140	110	110	140	150	130	120	100	140	150	150	100	130
Suspended solids, mg/l	110	100	94	72	62	58	62	72	55	53	100	86	110	53	76
Volatile suspended solids, mg/l	91	70	88	52	60	32	52	48	53	53	81	82	91	32	64
Settleable solids, ml/l	0.2	0.2	0.1	0.2	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.2	<0.1	<0.1
Total solids, mg/l	820	800	890	830	790	730	760	700	730	700	750	750	890	700	770
Ammonia nitrogen, mg/l															
pH	7.6	7.5	7.7	7.5	7.5	7.6	7.5	7.4	7.6	7.4	7.4	7.3	7.7	7.3	7.5
Primary Treatment Removals, Percent															
BOD$_5$	7	31	30	42	21	30	29	41	29	44	30	21	44	7	28
Suspended solids	45	64	48	60	56	55	61	67	66	69	50	52	69	45	58
Trickling Filter Effluent Characteristics															
BOD$_5$, mg/l	73	45	84	75	80	80	97	62	86	48	96	77	96	45	75
Suspended solids, mg/l	120	200	100	80	110	54	56	54	58	56	120	120	200	54	95
Volatile suspended solids, mg/l	98	130	88	46	100	36	40	20	55	54	99	101	130	20	72
Ammonia nitrogen, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Trickling Filter Effluent Changes, Percent															
BOD$_5$ reduction	48	59	40	32	27	43	35	52	28	52	31	49	59	28	42
Suspended solids increase	9	100	6	11	17	-7	-10	-25	5	5	20	40	100	-25	25
Volatile suspended solids increase	8	86	6	-12	67	13	23	-58	4	7	22	23	86	-58	13

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[a] ft^3/day x 0.028 = m^3/day

[b] Based on six months

TABLE E-2C. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1970

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft³/day[a]	1.2	2.3	1.3	1.6	0.9	0.7	1.2	1.2	1.7	2.0	1.6	1.2	2.3	0.7	1.4
Chlorine added, mg/l	2.4	2.7	2.6	0	0	0	2.4	2.2	2.6	2.8	0	0	2.8	0	2.5[b]
Primary Treatment Effluent Characteristics															
BOD_5, mg/l	180	180	130	140	140	110	140	110	100	140	140	130	180	100	140
Suspended solids, mg/l	160	110	89	96	88	55	51	71	56	61	91	73	160	51	83
Volatile suspended solids, mg/l	140	100	72	71	79	39	37	63	48	45	73	54	140	37	69
Settleable solids, ml/l	0.2	0.6	0.1	<0.1	<0.1	0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.6	<0.1	<0.2
Total solids, mg/l	820	870	820	810	770	760	750	750	680	740	730	840	870	680	780
Ammonia nitrogen, mg/l	48	55	40	49	49	45	36	43	33	43	40	33	55	33	43
pH	7.3	7.4	7.7	7.4	7.5	7.6	7.4	7.3	7.5	7.6	7.4	7.3	7.7	7.3	7.5
Primary Treatment Removals, Percent[c]															
BOD_5	22	25	40	44	42	50	36	58	50	44	46	48	58	22	42
Suspended solids	45	63	60	50	62	72	77	66	75	71	65	65	77	45	64
Trickling Filter Effluent Characteristics															
BOD_5, mg/l	110	140	130	130	96	70	110	68	66	85	110	82	140	66	100
Suspended solids, mg/l	100	90	81	110	88	75	56	67	52	65	97	89	110	52	81
Volatile suspended solids, mg/l	87	90	68	88	66	51	25	57	52	32	80	52	90	25	62
Ammonia nitrogen, mg/l	44	44	38	44	44	42	26	31	29	36	35	26	44	26	37
Trickling Filter Effluent Changes, Percent															
BOD_5 reduction	39	22	0	7	31	36	21	38	34	39	21	37	39	0	29
Suspended solids increase	-37	-18	-9	15	0	36	10	-6	-7	7	7	22	36	-37	-2
Volatile suspended solids increase	-38	-10	-6	24	-16	31	-32	-10	8	-29	10	-4	31	-38	-10
Ammonia nitrogen reduction	8	20	5	10	10	7	28	28	12	16	12	21	28	7	14

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[a] ft³/day x 0.028 = m³/day

[b] Based on seven months

[c] Affected by waste mixed liquor recycled to headworks

TABLE E-2D. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1971

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft³/day[a]	1.6	2.0	1.6	1.0	1.9	1.7	1.1	1.4	1.6	1.1	1.0	1.2	2.0	1.0	1.5
Chlorine added, mg/l	0	0	2.5	2.5	2.5	2.3	2.5	2.4	1.2	2.3	2.4	4.1	4.1	0	2.6[b]
Primary Treatment Effluent Characteristics															
BOD_5, mg/l	140	130	160	120	120	110	120	110	110	110	150	150	160	110	130
Suspended solids, mg/l	80	85	84	76	98	81	71	81	60	75	63	94	98	60	79
Volatile suspended solids, mg/l	65	66	60	53	71	54	33	71	44	56	57	88	88	33	61
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Total solids, mg/l	860	840	850	760	790	740	760	510	740	650	700	760	860	510	740
Ammonia nitrogen, mg/l	38	51	48	42	39	34	34	36	34	40	44	49	51	34	41
pH	6.9	7.4	7.3	7.4	7.3	7.3	7.5	6.9	6.9	7.1	7.4	7.4	7.5	6.9	7.2
Primary Treatment Removals, Percent[c]															
BOD_5	42	41	33	50	48	42	40	35	39	42	32	37	50	32	39
Suspended solids	60	57	62	60	59	69	60	69	60	60	70	61	70	57	65
Trickling Filter Effluent Characteristics															
BOD_5, mg/l	130	70	120	110	110	74	81	66	82	66	110	100	130	46	93
Suspended solids, mg/l	140	120	130	96	120	110	100	120	120	74	89	110	140	74	110
Volatile suspended solids, mg/l	117	92	106	89	94	89	100	101	80	63	73	100	117	63	92
Ammonia nitrogen, mg/l	34	41	42	33	31	27	23	25	22	33	36	44	44	22	33
Trickling Filter Effluent Changes, Percent															
BOD_5 reduction	7	39	25	8	8	33	32	50	25	40	27	33	50	7	28
Suspended solids increase	75	41	55	30	22	36	41	48	100	-1	41	17	100	-1	39
Volatile suspended solids increase	80	39	54	68	32	65	100	42	82	13	28	14	100	13	51
Ammonia nitrogen reduction	11	20	12	21	21	21	32	31	35	17	18	10	32	10	20

[a] ft³/day x 0.028 = m³/day

[b] Based on ten months

[c] Affected by waste mixed liquor recycled to headworks

TABLE E-2E. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1972

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft³/day[a]	1.1	1.8	2.0	1.8	1.4	1.1	2.0	2.7	2.6	2.3	1.4	1.6	2.7	1.1	1.8
Chlorine added, mg/l	4.9	4.1	3.8	3.5	3.6	4.4	4.5	4.6	4.5	5.4	4.2	3.4	5.4	3.4	4.3
Primary Treatment Effluent Characteristics															
BOD_5, mg/l	170	140	110	170	100	130	90	100	120	140	120	140	170	90	130
Suspended solids, mg/l	130	110	110	80	100	140	130	120	120	100	100	80	130	80	110
Volatile suspended solids, mg/l	100	88	110	66	81	110	36	88	62	80	70	85	110	36	81
Settleable solids, ml/l	<0.1	<0.1	0.4	0.1	<0.1	1.7	2.2	1.2	1.3	14.0	0.1	0.1	14.0	<0.1	1.8
Total solids, mg/l	810	800	700	710	690	760	820	800	730	610	720	850	850	610	760
Ammonia nitrogen, mg/l	41	44	-	42	40	24	20	30	35	40	38	44	44	20	36
pH	7.5	7.4	7.3	7.6	7.2	8.6	8.3	7.9	7.8	7.3	7.5	7.3	8.6	7.2	7.6
Primary Treatment Removals, Percent[b]															
BOD_5	25	42	[illegible]	43	44	54	59	50	40	30	40	33	59	25	43
Suspended solids	62	58	[illegible]	65	52	42	54	59	52	60	64	68	68	42	56
Trickling Filter Effluent Characteristics															
BOD_5, mg/l	110	110	130	130	75	120	67	120	100	130	100	130	130	67	110
Suspended solids, mg/l	130	140	120	160	110	86	84	140	120	120	130	130	160	84	120
Volatile suspended solids, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l	40	40	-	30	33	25	18	26	40	37	36	41	41	18	34
Trickling Filter Effluent Changes, Percent															
BOD_5 reduction	35	21	9	24	25	8	26	-20	17	7	17	7	35	-20	15
Suspended solids increase	0	27	9	100	10	-39	-35	17	0	20	30	63	100	-39	9
Ammonia nitrogen reduction	2	9	-	10	17	-4	10	13	-14	7	5	7	17	-14	6

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[a] ft³/day x 0.028 = m³/day

[b] Affected by waste mixed liquor recycled to headworks

TABLE E-2F. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1973

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft^3/day[a]	3.1	3.2	3.5	0.9	4.0	2.7	9.0	8.5	4.0	4.9	3.8	2.7	9.0	0.9	4.2
Chlorine added, mg/l	2.9	2.7	2.6	5.2	5.0	4.2	4.1	4.1	4.1	4.1	3.7	3.5	5.2	2.6	3.8
Primary Treatment Effluent Characteristics															
BOD$_5$, mg/l	120	140	120	140	130	130	130	110	110	130	120	130	140	110	120
Suspended solids, mg/l	100	88	81	92	72	80	95	73	80	53	74	69	100	53	80
Volatile suspended solids, mg/l	88	32	70	53	64	66	77	37	39	41	63	46	88	32	56
Settleable solids, ml/l	0.1	<0.1	0.1	0.1	<0.1	<0.1	<0.1	0.1	<0.1	0.2	0.2	<0.1	0.2	<0.1	<0.1
Total solids, mg/l	920	890	960	940	820	790	800	790	740	730	780	760	960	730	830
Ammonia nitrogen, mg/l	-	-	48	68	64	74	-	29	27	54	-	-	74	27	52
pH	7.5	7.4	7.5	7.4	7.3	7.3	7.3	7.3	7.4	7.4	7.3	7.3	7.5	7.3	7.3
Primary Treatment Removals, Percent[b]															
BOD$_5$	43	36	50	42	41	38	38	50	42	38	45	43	50	36	45
Suspended solids	52	62	65	67	70	70	59	72	70	71	70	70	72	52	67
Trickling Filter Effluent Characteristics															
BOD$_5$, mg/l	86	98	89	93	130	98	91	81	94	120	100	93	130	86	98
Suspended solids, mg/l	99	120	96	98	100	100	120	90	110	84	91	79	120	79	100
Volatile suspended solids, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l	-	-	46	60	54	60	-	21	22	57	-	-	60	21	46
Trickling Filter Effluent Changes, Percent															
BOD$_5$ reduction	28	30	26	34	0	25	30	26	15	8	17	28	34	0	18
Suspended solids increase	-1	36	10	7	39	25	26	23	38	58	23	14	58	-1	25
Ammonia nitrogen reduction	-	-	4	12	16	19	-	28	19	-6	-	-	28	-6	12

[a] ft^3/day x 0.28 = m^3/day

[b] Affected by waste mixed liquor recycled to headworks

TABLE E-2G. PRELIMINARY TREATMENT, PRIMARY TREATMENT, AND TRICKLING FILTER TREATMENT, 1974

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Preliminary Treatment															
Grit removed, ft^3/day^a	4.1	4.2	4.8	4.1	3.4	4.6	4.9	4.2	5.0	3.8	3.8	5.0	5.0	3.4	5.3
Chlorine added, mg/l	3.6	5.5	6.6	7.1	7.5	8.6	8.4	7.0	6.7	6.5	6.5	6.5	8.6	3.6	6.7
Primary Treatment Effluent Characteristics															
BOD_5, mg/l	150	160	160	130	100	140	140	120	120	120	120	-	160	100	130
Suspended solids, mg/l	100	50	77	85	72	79	74	87	76	79	86	-	100	50	79
Volatile suspended solids, mg/l	87	55	64	62	53	57	64	73	60	76	66	-	87	53	65
Settleable solids, ml/l	0.1	0.2	<0.1	0.2	0.4	0.4	0.1	0.1	0.1	<0.1	0.2	-	0.4	<0.1	0.2
Total solids, mg/l	830	860	850	830	820	750	710	750	660	650	590	-	860	590	760
Ammonia nitrogen, mg/l	-	-	-	-	-	21	-	-	45	42	57	-	57	21	41
pH	7.4	7.5	7.4	7.3	7.4	7.2	7.3	7.3	7.1	7.2	7.1	-	7.5	7.1	7.3
Primary Treatment Removals, Percent[b]															
BOD_5	32	30	36	35	58	36	30	45	40	37	29	-	58	29	38
Suspended solids	67	78	67	72	64	64	69	69	65	64	69	-	78	64	68
Trickling Filter Effluent Characteristics															
BOD_5, mg/l	160	$56^c/160$	92	$26^c/59$	$43^c/85$	$42^c/97$	$41^c/69$	$23^c/61$	$42^c/70$	$43^c/62$	$28^c/52$	-	$56^c/160$	$23^c/52$	$38^c/88$
Suspended solids, mg/l	150	170	84	91	120	75	85	56	68	100	72	-	170	56	97
Volatile suspended solids, mg/l	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l	-	-	-	-	-	19	-	-	35	39	52	-	52	19	36
Trickling Filter Effluent Changes, Percent															
BOD_5 reduction	-7	$65^c/0$	43	$80^c/55$	$57^c/15$	$70^c/31$	$71^c/51$	$81^c/49$	$65^c/42$	$64^c/48$	$77^c/57$	-	$81^c/57$	$57^c/-7$	$71^c/32$
Suspended solids increase	50	240	9	7	67	-5	18	-36	-11	27	16	-	240	-36	23
Ammonia nitrogen reduction	-	-	-	-	-	10	-	-	22	7	9	-	22	7	12

[a] $ft^3/day \times 0.28 = m^3/day$

[b] Affected by waste mixed liquor recycled to headworks

[c] Based on one-hour settling

TABLE E-3A. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1968

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft^3/day[a]	2.9	3.4	4.7	4.4	4.9	5.2	5.3	6.7	8.3	8.5	8.6	8.3	8.6	2.9	5.9
Dissolved oxygen, mg/l	3.9	3.3	2.3	4.2	4.6	4.9	4.6	4.0	4.2	4.1	4.3	4.0	4.7	2.3	4.0
Mixed liquor suspended solids, mg/l	2,150	1,880	1,900	1,690	1,500	1,450	1,560	1,620	1,760	1,610	1,880	1,580	2,150	1,450	1,720
Settleable solids, ml/l	207	206	203	162	134	113	118	109	121	101	122	109	207	101	142
Sludge volume index (SVI)	97	109	107	96	90	77	75	68	70	62	65	69	109	62	82
Waste mixed liquor flow, 1,000 gpd[b]	223	191	194	186	120	158	155	187	278	100	237	228	278	100	188
Settled activated sludge suspended solids, mg/l	7,630	5,450	6,450	7,150	7,880	8,250	8,586	8,880	9,180	7,680	7,570	6,800	9,180	5,450	7,620
Return activated sludge flow, percent of influent flow	45	57	43	35	31	27	28	28	31	35	37	31	57	27	36
Secondary Effluent Characteristics	SECONDARY EFFLUENT NOT MONITORED IN 1968														
BOD$_5$, mg/l															
Suspended solids, mg/l															
Volatile suspended solids, mg/l															
Settleable solids, ml/l															
Dissolved oxygen, mg/l															
pH															
Ammonia nitrogen, mg/l															
Nitrite nitrogen, mg/l															
Nitrate nitrogen, mg/l															
Activated Sludge-Secondary Clarification Removals, Percent															
BOD$_5$															
Suspended solids															
Ammonia nitrogen															

[a] million ft^3/day x 28,000 = m^3/day

[b] gpd x 0.0038 = m^3/day

TABLE E-3B. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1969

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft^3/day[a]	6.1	7.7	7.7	8.0	8.0	8.1	6.7	6.1	6.9	6.4	6.4	6.8	8.1	6.1	7.1
Dissolved oxygen, mg/l	3.2	4.4	4.4	3.5	3.4	3.5	2.5	2.5	2.3	2.8	2.8	2.2	4.4	2.2	3.1
Mixed liquor suspended solids, mg/l	1,730	1,700	1,690	1,700	1,760	1,790	1,800	1,740	1,740	1,680	1,790	1,880	1,880	1,680	1,750
Settleable solids, ml/l	109	71	92	102	141	157	152	140	142	157	144	119	157	71	127
Sludge volume index (SVI)	65	42	55	60	80	87	85	81	82	94	80	62	94	42	73
Waste mixed liquor flow, 1,000 gpd[b]	223	206	190	122	147	146	146	141	178	129	132	178	223	122	162
Settled activated sludge suspended solids, mg/l	7,190	6,830	6,460	6,220	6,570	6,370	5,990	5,660	5,490	5,590	6,180	5,640	7,190	5,490	6,180
Return activated sludge flow, percent of influent flow	28	24	27	29	33	33	32	33	35	36	32	41	41	24	32
Secondary Effluent Characteristics															
BOD_5, mg/l	SECONDARY EFFLUENT NOT MONITORED IN 1969														
Suspended solids, mg/l															
Volatile suspended solids, mg/l															
Settleable solids, ml/l															
Dissolved oxygen, mg/l															
pH															
Ammonia nitrogen, mg/l															
Nitrite nitrogen, mg/l															
Nitrate nitrogen, mg/l															
Activated Sludge-Secondary Clarification Removals, Percent															
BOD_5															
Suspended solids															
Ammonia nitrogen															

[a] million ft^3/day x 28,000 = m^3/day

[b] gpd x 0.0038 = m^3/day

TABLE E-3C. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1970

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft³/day[a]	7.7	7.5	7.9	7.4	7.1	7.1	7.2	7.4	6.2	5.8	6.0	6.3	7.9	5.8	7.0
Dissolved oxygen, mg/l	3.2	3.1	3.2	2.9	3.0	3.0	2.9	3.1	3.1	3.0	3.3	3.2	3.3	2.9	3.1
Mixed liquor suspended solids, mg/l	2,060	2,210	2,660	2,380	2,550	2,550	2,620	2,650	2,450	1,800	1,700	1,870	2,660	1,700	2,290
Settleable solids, ml/l	146	110	156	179	242	256	217	250	274	187	146	147	274	110	193
Sludge volume index (SVI)	73	50	59	74	95	100	83	98	113	104	86	79	113	50	85
Waste mixed liquor flow, 1,000 gpd[b]	200	80	96	66	130	90	94	100	130	128	129	175	200	66	118
Settled activated sludge suspended solids, mg/l	5,920	6,850	8,700	7,750	8,060	8,380	9,160	8,820	8,130	6,360	6,540	7,670	9,160	5,920	7,700
Return activated sludge flow, percent of influent flow	44	45	38	38	36	39	37	36	37	33	32	30	45	30	37
Secondary Effluent Characteristics															
BOD_5, mg/l	19	29	17	18	18	12	12	7.0	9.5	12	6.6	10	29	6.6	14
Suspended solids, mg/l	17	44	14	23	16	18	9	15	13	14	21	20	44	9	19
Volatile suspended solids, mg/l	14	38	10	12	11	6	4	9	13	9	18	16	38	4	13
Settleable solids, ml/l	0.1	0.1	0.4	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.4	<0.1	<0.1
Dissolved oxygen, mg/l	2.3	2.4	3.3	3.1	3.6	4.2	3.3	4.4	4.0	3.5	3.0	2.8	4.4	2.3	3.3
pH	6.8	7.0	7.5	7.3	7.2	7.6	7.3	7.3	7.4	7.3	7.5	7.3	7.6	6.8	7.3
Ammonia nitrogen, mg/l	1.9	1.1	1.1	0.77	1.1	0.95	0.43	1.3	0.63	1.0	0.76	0.31	1.9	0.31	0.95
Nitrite nitrogen, mg/l	0.54	0.037	0.052	0.033	0.064	0.046	0.005	0.010	0.010	0.033	0.057	0.081	0.540	0.005	0.081
Nitrate nitrogen, mg/l	24	21	22	21	21	22	21	17	18	21	20	19	24	17	21
Activated Sludge-Secondary Clarification Removals, Percent															
BOD_5	83	79	87	86	81	83	89	90	84	86	94	88	94	79	86
Suspended solids	83	51	83	79	82	76	84	78	75	78	78	78	84	51	77
Ammonia nitrogen	96	97	97	98	97	98	98	99	98	97	98	99	99	96	97

[a] million ft³/day x 28,000 = m³/day

[b] gpd x 0.0038 = m³/day

TABLE E-3D. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1971

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft³/day[a]	6.4	6.4	5.9	8.0	6.3	6.8	6.9	7.2	7.3	7.3	7.1	6.5	8.0	5.9	6.8
Dissolved oxygen, mg/l	3.1	3.1	3.4	2.5	3.5	3.0	2.9	2.5	2.5	2.7	3.1	3.0	3.5	2.5	2.9
Mixed liquor suspended solids, mg/l	1,800	1,760	1,700	1,750	1,740	2,110	2,190	2,160	2,120	2,160	2,280	2,320	2,320	1,700	2,010
Settleable solids, ml/l	185	114	142	144	159	168	172	172	156	140	133	121	185	121	151
Sludge volume index (SVI)	99	66	79	82	91	79	78	79	74	64	58	52	99	52	75
Waste mixed liquor flow, 1,000 gpd[b]	217	197	211	226	179	160	112	101	103	83	141	176	226	83	159
Settled activated sludge suspended solids, mg/l	7,470	8,160	6,870	6,850	6,650	8,240	8,280	8,000	8,280	8,380	8,270	8,800	8,800	6,650	7,890
Return activated sludge flow, percent of influent flow	30	30	30	30	30	33	38	37	38	37	37	38	38	30	34
Secondary Effluent Characteristics															
BOD_5, mg/l	8.9	12	17	8.7	12	6.6	9.1	5.2	5.4	11	11	13	17	5.2	10
Suspended solids, mg/l	20	17	24	15	19	24	19	31	19	18	31	27	31	15	22
Volatile suspended solids, mg/l	15	11	16	10	10	15	7	20	9	14	20	24	24	7	14
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.4	1.1	0.6	0.2	1.3	<0.1	1.3	<0.1	<0.4
Dissolved oxygen, mg/l	2.3	2.3	2.6	2.8	3.7	3.5	3.6	3.7	3.3	2.9	3.1	2.9	3.2	2.3	3.1
pH	7.1	7.5	7.5	7.5	7.4	7.6	7.8	7.8	7.3	7.3	7.7	7.4	7.8	7.1	7.5
Ammonia nitrogen, mg/l	0.86	1.1	6.7	0.94	0.76	0.48	0.88	0.88	1.75	1.03	0.46	4.3	6.7	0.46	1.7
Nitrite nitrogen, mg/l	0.065	0.32	0.78	0.018	0.046	0.017	0.040	0.12	0.17	0.030	0.10	0.13	0.78	0.017	0.077
Nitrate nitrogen, mg/l	18	18	19	19	20	18	18	19	18	18	19	17	20	17	18
Activated Sludge-Secondary Clarification Removals, Percent															
BOD_5	93	85	86	92	89	91	89	87	93	83	90	87	93	85	89
Suspended solids	86	86	82	84	84	78	81	74	84	76	65	75	86	65	80
Ammonia nitrogen	97	97	84	97	98	98	96	96	92	97	99	90	99	84	95

[a] million ft³/day x 28,000 = m³/day

[b] gpd x 0.0038 = m³/day

TABLE E-3E. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1972

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft^3/day[a]	6.7	7.8	7.5	7.5	7.6	7.6	7.6	7.6	7.8	8.0	8.8	8.9	8.9	6.7	7.8
Dissolved oxygen, mg/l	2.3	2.5	2.4	1.9	0.9	0.5	0.6	0.4	0.3	0.3	0.3	0.2	2.5	0.2	1.1
Mixed liquor suspended solids, mg/l	2,240	2,480	2,340	2,550	2,490	2,390	2,500	2,550	2,690	2,890	2,840	3,010	3,010	2,240	2,580
Settleable solids, ml/l	193	232	156	172	169	153	197	211	254	282	224	238	282	153	207
Sludge volume index (SVI)	83	94	66	67	68	64	79	83	94	98	79	99	98	64	80
Waste mixed liquor flow, 1,000 gpd[b]	172	196	107	160	131	117	122	171	156	152	170	176	196	107	153
Settled activated sludge suspended solids, mg/l	8,540	8,500	8,010	8,560	8,530	8,080	8,120	8,780	9,160	8,150	8,450	9,390	9,390	8,010	8,520
Return activated sludge flow, percent of influent flow	37	39	38	39	37	37	-	36	36	40	43	42	43	36	39
Secondary Effluent Characteristics															
BOD$_5$, mg/l	17	15	14	12	24	14	3.6	7.1	19	19	12	22	24	3.6	15
Suspended solids, mg/l	39	50	77	34	44	39	30	20	40	32	33	38	77	20	40
Volatile suspended solids, mg/l	21	29	57	33	27	17	26	9	21	28	27	28	57	9	27
Settleable solids, ml/l	0.2	0.5	0.2	0.5	0.1	0.6	1.9	0.9	1.9	0.1	<0.1	0.1	1.9	<0.1	0.6
Dissolved oxygen, mg/l	2.0	2.3	2.5	2.4	2.3	2.2	2.9	2.7	2.7	2.3	2.4	2.5	2.9	2.0	2.4
pH	7.3	7.2	7.4	7.8	7.3	8.4	8.1	8.0	7.9	7.6	7.4	7.3	8.4	7.2	7.6
Ammonia nitrogen, mg/l	1.2	1.1	-	0.81	1.1	1.1	0.77	1.1	1.5	1.2	0.48	1.1	1.2	0.77	1.0
Nitrite nitrogen, mg/l	0.23	0.17	-	0.09	0.02	0.10	0.03	0.09	0.13	0.01	0.01	0.22	0.23	0.01	0.10
Nitrate nitrogen, mg/l	19	21	-	16	20	17	15	17	14	20	20	18	21	14	18
Activated Sludge-Secondary Clarification Removals, Percent															
BOD$_5$	85	86	87	91	68	88	95	94	81	85	88	83	95	68	86
Suspended solids	70	64	36	79	60	55	64	86	67	73	75	71	86	36	67
Ammonia nitrogen	97	97	-	98	97	96	96	96	96	97	99	97	99	96	97

[a] million ft^3/day x 28,000 = m^3/day

[b] gpd x 0.0038 = m^3/day

TABLE E-3F. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1973

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge Aeration Tank															
Air used, million ft^3/day[a]	8.8	8.8	8.8	8.7	8.7	8.8	8.8	8.9	8.8	8.8	8.9	8.6	8.9	8.6	8.8
Dissolved oxygen, mg/l	0.2	0.2	0.2	0.4	0.9	0.8	0.9	0.7	0.5	1.0	0.5	0.6	1.0	0.2	0.5
Mixed liquor suspended solids, mg/l	2,770	2,430	2,540	2,260	2,360	2,280	2,190	2,240	2,060	2,290	2,460	2,450	2,770	2,060	2,360
Settleable solids, ml/l	196	158	128	105	133	150	166	176	144	147	154	136	196	105	150
Sludge volume index (SVI)	71	66	50	46	57	68	76	78	70	65	63	56	78	46	64
Waste mixed liquor flow, 1,000 gpd[b]	190	160	144	128	125	142	98	170	160	196	172	147	196	98	154
Settled activated sludge suspended solids, mg/l	8,710	7,780	8,340	7,580	8,060	7,650	7,250	7,440	7,760	7,730	8,540	8,480	8,710	7,250	7,940
Return activated sludge flow, percent of influent flow	38	35	37	36	37	40	39	38	39	36	-	-	40	35	38
Secondary Effluent Characteristics															
BOD$_5$, mg/l	17	15	15	22	15	8.6	19	7.8	14	13	19	24	24	7.8	16
Suspended solids, mg/l	35	31	29	29	27	25	20	34	35	13	21	42	42	13	28
Volatile suspended solids, mg/l	32	14	20	18	23	20	19	7	18	12	15	20	32	7	18
Settleable solids, ml/l	1.8	0.1	0.2	0.1	<0.1	<0.1	<0.1	0.2	<0.1	0.2	<0.1	<0.1	1.8	<0.1	0.3
Dissolved oxygen, mg/l	2.8	2.3	2.0	2.5	2.5	3.0	2.6	2.7	2.3	2.9	2.0	2.2	3.0	2.0	2.5
pH	7.5	7.2	7.2	7.3	7.4	7.3	7.2	7.2	7.3	7.3	7.2	7.0	7.5	7.0	7.3
Ammonia nitrogen, mg/l	-	-	0.49	1.6	1.2	1.1	-	1.1	1.8	3.8	1.1	-	3.8	0.49	1.5
Nitrite nitrogen, mg/l	-	-	-	0.31	0.06	0.04	-	0.05	0.06	0.40	<0.01	-	0.40	<0.01	0.13
Nitrate nitrogen, mg/l	-	-	16	20	18	18	-	14	14	12	13	-	20	12	16
Activated Sludge-Secondary Clarification Removals, Percent															
BOD$_5$	80	85	83	76	88	91	79	90	85	89	81	74	91	74	84
Suspended solids	65	74	70	70	73	75	83	62	68	85	77	47	85	47	72
Ammonia nitrogen	-	-	99	97	98	98	-	95	92	93	-	-	99	92	97

[a] million ft^3/day × 28,000 = m^3/day

[b] gpd × 0.0038 = m^3/day

TABLE E-3G. ACTIVATED SLUDGE AERATION TANK AND SECONDARY TREATMENT, 1974

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Activated Sludge															
Aeration Tank															
Air used, million ft^3/day[a]	8.6	5.9	8.8	8.9	8.4	8.2	8.3	8.3	8.8	8.7	8.7	8.2	8.9	5.9	8.3
Dissolved oxygen, mg/l	1.3	0.6	1.6	1.5	1.1	0.6	0.8	0.6	0.4	0.8	0.9	1.2	1.6	0.4	1.0
Mixed liquor suspended solids, mg/l	2,330	2,510	2,450	2,260	1,970	2,030	2,060	1,870	1,820	1,970	2,000	2,090	2,510	1,820	2,110
Settleable solids, ml/l	121	158	176	129	138	132	137	124	117	143	138	195	195	117	142
Sludge volume index (SVI)	52	62	72	57	70	65	66	69	65	73	70	93	93	52	68
Waste mixed liquor flow, 1,000 gpd[b]	152	129	144	170	173	173	173	173	173	173	173	173	173	129	164
Settled activated sludge suspended solids, mg/l	8,300	8,470	7,530	7,520	7,250	7,550	7,670	7,350	7,160	6,280	6,260	5,100	8,470	5,100	7,200
Return activated sludge flow, percent of influent flow	33	38	45	39	35	40	34	33	32	41	40	59	59	32	39
Secondary Effluent Characteristics															
BOD_5, mg/l	23	40	21	24	25	16	13	9.1	12	11	14	-	40	9.1	19
Suspended solids, mg/l	51	14	21	31	29	25	16	10	13	27	21	-	51	10	24
Volatile suspended solids, mg/l	32	10	10	18	23	17	12	8	9	27	13	-	32	8	17
Settleable solids, ml/l	0.3	0.4	<0.1	0.2	<0.1	<0.1	0.1	0.3	0.1	0.1	0.2	-	0.4	<0.1	0.2
Dissolved oxygen, mg/l	2.0	1.6	3.6	2.3	3.1	3.0	2.3	2.7	2.1	2.0	1.7	2.0	3.6	1.6	2.4
pH	7.1	7.3	7.1	7.0	7.2	6.9	7.2	7.2	7.0	7.0	6.9	-	7.3	6.9	7.1
Ammonia nitrogen, mg/l	-	-	-	-	-	<0.1	-	-	2.5	2.5	1.6	-	2.5	<0.1	1.7
Nitrite nitrogen, mg/l	-	-	-	-	-	1.0	-	-	0.12	0.03	0.140	-	1.0	0.03	0.32
Nitrate nitrogen, mg/l	-	-	-	-	-	17	-	-	16	20	20	-	20	16	18
Activated Sludge-Secondary Clarification Removals, Percent															
BOD_5	86	75	77	59	71	84	81	85	83	82	73	-	86	59	78
Suspended solids	66	92	75	66	76	67	81	82	81	73	71	-	92	66	75
Ammonia nitrogen	-	-	-	-	-	99	-	-	93	94	97	-	99	93	95

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[a] million ft^3/day x 28,000 = m^3/day

[b] gpd x 0.0038 = m^3/day

TABLE E-4A. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1968

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	-	31	29	36	18	17	17	16	19	18	20	20	36	16	20
Contact time, min.	96	117	-	116	93	92	89	89	88	95	122[a]	92	122	88	99
Chlorine residual, mg/l	7.0	8.7	7.1	6.6	7.1	>8.7[d]	>6.0[d]	>6.2[d]	>6.3[d]	>5.4[d]	>8.7[d]	>8.8[d]	8.8[d]	6.0[d]	77.2[d]
pH, contact tank effluent	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Median coliform organism concentration, MPN/100ml	4.8	2.1	4.8	<2.1	4.8	2.1	2.1	2.1	<2.1	<2.1	<2.1	<2.1	4.8	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	17	15	10	9.0	18	15	16	0.9	0.1	3.1	8.1	3.1	18	0.1	9.6
Suspended solids, mg/l	7.0	8.6	10	13	12	8.0	5.0	9.5	7.0	11	14	8.0	14	5.0	9.5
Volatile suspended solids, mg/l	6.0	8.2	4.0	8.0	11	7.0	3.8	7.1	5.3	10.3	9.3	7.8	10.8	4.0	7.3
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	1.1	1.1	2.3	4.3	4.4	4.7	4.2	2.8	2.1	2.3	2.1	2.9	4.7	1.1	2.9
pH	7.9	7.6	7.8	7.6	7.0	7.6	7.8	7.6	7.8	7.6	6.6	6.8	7.9	6.6	7.5
Grease, mg/l	8.0	7.4	3.3	3.4	4.4	2.5	1.0	1.8	6.0	2.8	3.1	4.1	8.0	1.0	4.0
MBAS, mg/l	0.50	0.41	0.53	0.44	0.40	0.40	0.44	0.45	0.36	0.26	0.21	0.30	0.53	0.21	0.39
Turbidity, JTU	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature, C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l[a]	16	7.5	<0.01	28	1.0	<0.01	0.48	0.54	0.33	0.12	-	0.23	28	<0.01	4.9
Nitrite nitrogen, mg/l[a]	0.06	0.09	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	0.03	-	0.01	0.09	<0.01	0.02
Nitrate nitrogen, mg/l[a]	7.1	7.6	15	8.0	14	15	16	14	21	26	-	11	26	7.1	14
Organic nitrogen, mg/l[a]	0.8	2.5	2.9	3.5	0.7	1.2	-	-	2.1	1.7	-	0.63	3.5	0.63	1.8
Total solids, mg/l	1,110	1,140	1,300	1,470	1,270	1,120	1,090	1,080	1,020	850	800	760	1,470	760	1,080
Total dissolved solids, mg/l	840	1,140	1,290	1,460	1,260	1,110	1,090	1,050	1,010	840	790	760	1,460	760	1,050
Fixed dissolved solids, mg/l	610	980	1,010	1,140	1,020	860	870	880	610	600	560	550	1,140	550	800
Specific conductivity, micromhos	1,550	1,630	1,660	2,180	1,920	1,700	1,720	1,720	1,220	1,160	1,000	1,150	2,180	1,000	1,550
Chlorides, mg/l	390	330	450	440	470	360	350	360	350	240	230	250	470	230	350
Dissolved residue (by calculation), mg/l[a]	1,030	1,150	1,200	-	1,080	1,050	1,020	300	-	-	-	-	1,200	300	980
Alkalinity, mg/l[a]	170	110	110	140	100	120	120	96	69	70	-	-	170	69	110
Hardness, mg/l[a]	320	280	350	320	300	270	250	240	240	220	-	-	350	220	280
Boron, mg/l[a]	1.0	1.6	1.3	2.2	2.1	2.5	1.1	1.4	1.0	1.4	-	-	2.5	1.0	1.6
Percent sodium[a]	60	62	63	60	64	64	64	63	65	64	-	-	65	54	52
Overall Plant Removals, Percent															
BOD_5	90	91	95	95	90	92	92	>99	>99	98	95	90	>99	90	95
Suspended solids[b]	96	95	97	95	95	97	98	97	96	93	92	95	98	92	95
Ammonia nitrogen[b]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

[d] Value affected by maximum meter reading of 10 mg/l

TABLE E-4B. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1969

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	19	18	20	20	25	21	-	-	-	19	-	19	25	18	20
Contact time, min.	87	56	67	62	61	59	58	56	60	67	72	68	87	56	64
Chlorine residual, mg/l[d]	>7.9	>5.8	>8.1	>6.3	>10.0	>9.5	>8.1	>10.0	>10.0	>8.9	>9.6	>7.4	>10.0	>5.8	>8.5
pH, contact tank effluent	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Median coliform organism concentration, MPN/100ml	<2.1	2.1	<2.1	2.1	<2.1	4.8	2.1	4.8	2.1	2.1	3.4	12	12	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	14	7.0	5.5	6.1	3.1	5.0	7.1	5.1	8.0	8.1	4.8	6.1	14	3.1	6.6
Suspended solids, mg/l	19	17	12	13	6	6	4	5	5	9	21	25	25	4	12
Volatile suspended solids, mg/l	14	10	7	9	6	2	2	3	5	9	14	18	18	2	8
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	2.5	2.8	3.0	2.0	2.2	2.6	2.6	2.9	2.3	2.6	4.7	4.8	4.8	2.0	2.9
pH	7.3	7.4	7.4	7.1	7.1	7.2	7.6	7.4	7.1	6.9	7.3	7.3	7.6	6.9	7.3
Grease, mg/l	5.5	3.5	2.0	6.4	1.9	0.4	1.9	4.0	1.7	0.8	1.1	2.0	6.4	0.4	2.6
MBAS, mg/l	0.25	0.09	0.27	0.15	0.16	0.10	0.08	0.07	0.12	0.13	0.24	0.10	0.27	0.07	0.15
Turbidity, JTU	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature, C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l[a]	4.1	0.78	0.11	0.03	0.12	<0.02	0.06	<0.02	0.06	<0.01	0.47	0.33	4.1	<0.01	0.51
Nitrite nitrogen, mg/l[a]	0.13	0.01	0.01	0.002	0.001	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	0.02	0.13	<0.01	<0.02
Nitrate nitrogen, mg/l[a]	12	19	26	25	24	28	19	19	21	24	27	25	28	12	22
Organic nitrogen, mg/l[a]	2.9	2.6	0.85	2.7	0.09	0.02	0.02	0.02	0.02	1.4	1.7	3.8	3.8	0.02	1.3
Total solids, mg/l	800	790	950	860	780	750	760	740	740	740	760	790	950	740	800
Total dissolved solids, mg/l	780	770	940	840	780	710	760	730	730	720	740	770	940	710	770
Fixed dissolved solids, mg/l	530	480	650	610	460	502	540	540	460	410	470	480	650	410	510
Specific conductivity, micromhos	1,010	960	1,140	1,220	1,160	980	1,020	1,000	1,000	1,000	900	920	1,220	900	1,030
Chlorides, mg/l	230	180	240	260	220	210	200	160	150	200	150	160	260	150	200
Dissolved residue (by calculation), mg/l[a]	740	700	880	830	760	750	740	700	700	680	750	760	880	680	750
Alkalinity, mg/l[a]	83	82	98	82	94	62	88	110	95	58	78	87	110	58	85
Hardness, mg/l[a]	230	200	250	250	140	190	210	210	210	180	200	200	250	180	210
Boron, mg/l[a]	1.6	1.5	1.8	2.1	1.6	1.4	1.4	1.3	1.5	1.3	1.5	1.3	2.1	1.3	1.5
Percent sodium[a]	57	58	58	59	59	69	-	60	58	63	58	65	69	57	60
Overall Plant Removals, Percent															
BOD_5	91	96	97	97	98	97	97	98	95	95	98	97	98	91	96
Suspended solids	90	94	93	93	96	95	97	98	97	97	89	86	98	86	93
Ammonia nitrogen[b]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

[d] Values affected by maximum meter reading of 10 mg/l

TABLE E-4C. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1970

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	25	28	20	24	24	20	21	18	24	27	25	25	28	18	25
Contact time, min.	68	77	70	69	72	63	73	69	72	76	73	70	78	63	72
Chlorine residual, mg/l[d]	>8.0	>5.0	>7.4	>10.0	>10.0	>10.0	>10.0	>9.5	>7.0	>7.9	>9.9	>9.7	>10.0	>5.0	>8.8
pH, contact tank effluent	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Median coliform organism concentration, MPN/100ml	2.1	4.8	2.1	2.1	3.4	4.8	<2.1	2.1	4.8	2.1	2.1	2.1	4.8	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	4.9	10	4.0	7.0	13	7.6	12	5.7	7.6	10	5.1	10	13	4.0	8.1
Suspended solids, mg/l	9	35	11	11	7	8	4	9	10	6	13	17	35	4	13
Volatile suspended solids, mg/l	8	20	5	6	5	3	2	7	10	4	11	13	20	2	8
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	4.6	4.7	5.7	5.1	5.9	5.7	4.5	5.7	5.3	5.1	4.9	5.0	5.9	4.5	5.2
pH	6.6	7.3	7.2	7.0	6.8	7.4	7.5	7.0	7.0	6.9	7.1	6.7	7.5	6.6	7.0
Grease, mg/l	5.8	4.0	1.9	1.3	1.4	0.2	3.2	2.9	2.7	1.0	1.0	3.3	5.8	0.2	2.5
MBAS, mg/l	0.06	0.11	0.07	0.08	0.17	0.03	0.09	0.06	0.04	0.04	0.04	0.04	0.17	0.03	0.07
Turbidity, JTU	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature, C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l[a]	<0.02	<0.02	0.44	<0.02	<0.02	0.2	<0.1	<0.1	<0.1	<0.1	nil	nil	0.44	<0.1	<0.11
Nitrite nitrogen, mg/l[a]	0.02	0.01	<0.01	<0.01	<0.01	<0.01	0.02	0.10	nil	nil	nil	nil	0.10	<0.01	<0.03
Nitrate nitrogen, mg/l[a]	20	26	26	19	23	21	18	18	20	24	24	22	26	18	22
Organic nitrogen, mg/l[a]	1.1	2.8	1.7	0.02	0.8	0.7	0.8	1.4	0.1	1.6	0.7	0.7	2.8	0.02	1.0
Total solids, mg/l	770	940	780	790	740	720	750	720	700	730	760	800	940	700	780
Total dissolved solids, mg/l	760	880	780	780	740	710	750	720	700	720	740	800	880	700	760
Fixed dissolved solids, mg/l	370	530	540	550	510	470	560	510	480	480	480	520	560	370	490
Specific conductivity, micromhos	900	1,080	940	1,080	1,030	900	1,090	1,010	960	920	810	1,060	1,090	810	980
Chlorides, mg/l	170	200	180	190	180	160	180	170	180	160	150	180	200	150	170
Dissolved residue (by calculation), mg/l[a]	680	820	710	730	690	660	660	660	640	630	-	710	820	630	680
Alkalinity, mg/l[a]	47	54	51	100	72	76	91	89	75	60	64	89	100	47	74
Hardness, mg/l[a]	180	200	200	240	190	190	210	180	190	180	210	230	240	180	200
Boron, mg/l[a]	1.4	1.6	1.6	1.4	1.3	1.2	0.9	1.4	1.4	0.8	0.1	1.3	1.6	0.1	1.2
Percent sodium[a]	62	61	61	54	62	50	56	61	50	56	55	54	62	54	58
Overall Plant Removals, Percent															
BOD_5	96	96	98	97	96	97	95	98	96	95	98	96	98	95	97
Suspended solids	97	88	95	95	97	96	98	96	95	97	95	93	98	88	95
Ammonia nitrogen[b]	>99	>99	99	>99	>99	>99	>99	>99	>99	>99	>99	>99	>99	99	>99

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

[d] Affected by maximum meter reading of 10 mg/l

TABLE E-4D. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1971

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	36	36	29	25	30	31	34	32	26	29	36	36	36	25	32
Contact time, min.	74	81	67	64	67	76	74	76	77	82	80	79	82	64	75
Chlorine residual, mg/l	9.5	16	7.8	8.7	16	15	20	18	13	14	13	16	20	7.8	14
pH, contact tank effluent	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Median coliform organism concentration, MPN/100ml	-	2.1	2.1	2.1	2.1	2.1	<2.1	<2.1	2.1	2.1	2.1	2.1	2.1	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	9.3	6.0	5.2	3.1	8.5	6.8	7.3	8.1	6.1	7.8	12	7.0	12	3.1	7.3
Suspended solids, mg/l	19	13	16	8	12	12	9	17	11	15	12	17	19	8	13
Volatile suspended solids, mg/l	6	3	6	3	5	5	6	5	5	2	3	2	6	2	4
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	3.5	3.6	4.2	3.9	5.1	5.0	5.0	4.8	4.6	4.1	4.2	4.5	5.1	3.6	4.4
pH	6.3	7.0	6.9	6.8	6.6	6.5	6.5	6.6	6.7	6.5	7.1	6.5	7.1	6.3	6.7
Grease, mg/l	1.8	2.5	2.3	2.3	1.8	3.3	1.8	2.2	2.2	2.3	1.8	4.6	4.6	1.5	2.3
MBAS, mg/l	0.05	0.04	0.04	0.05	0.05	0.05	0.03	0.03	0.04	0.03	0.05	0.04	0.05	0.03	0.04
Turbidity, JTU	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature, C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l[a]	<0.1	<0.1	1.3	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	0.4	1.3	<0.1	<0.2
Nitrite nitrogen, mg/l[a]	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.1	1.5	<0.01	<0.01	<0.01	<0.01	1.5	<0.01	<0.14
Nitrate nitrogen, mg/l[a]	23	14	20	19	30	22	17	18	24	21	25	25	30	14	22
Organic nitrogen, mg/l[a]	2.0	3.1	1.9	2.0	0.1	7.6	1.0	1.3	0.4	1.7	4.0	1.3	7.8	0.1	2.2
Total solids, mg/l	890	860	840	780	780	920	740	680	720	660	730	760	920	660	790
Total dissolved solids, mg/l	670	850	820	770	780	910	730	660	710	650	720	740	910	650	770
Fixed dissolved solids, mg/l	610	570	590	520	540	700	540	440	540	440	440	490	700	440	530
Specific conductivity, micromhos	1,110	1,010	1,100	980	940	1,030	1,000	910	1,020	770	910	940	1,110	770	980
Chlorides, mg/l	200	190	180	160	160	150	160	160	130	120	130	150	200	120	160
Dissolved residue (by calculation), mg/l[b]	720	730	720	660	700	570	640	620	620	570	620	650	730	570	660
Alkalinity, mg/l[a]	72	65	89	71	80	84	71	62	69	31	41	52	89	31	68
Hardness, mg/l[a]	220	220	240	200	200	230	200	180	170	150	170	190	240	150	200
Boron, mg/l[a]	1.4	2.1	1.5	1.1	-	1.2	1.3	0.5	0.8	0.5	0.5	0.9	2.1	0.5	1.1
Percent sodium[a]	51	58	52	60	56	58	56	55	59	-	60	57	60	52	57
Overall Plant Removals, Percent															
BOD_5	96	97	98	99	96	96	96	95	97	96	95	97	99	95	97
Suspended solids	92	93	93	97	93	95	96	93	94	94	94	93	97	92	94
Ammonia nitrogen	>99	>99	97	>99	>99	>99	>98	>99	>99	>99	>99	>99	>99	97	>99

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

TABLE E-4E. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1972

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	31	32	31	31	32	26	25	22	23	33	29	34	34	22	29
Contact time, min.	59	66	56	56	53	51	51	49	50	50	58	54	66	49	54
Chlorine residual, mg/l	18	20	16	15	16	14	17	15	11	15	16	18	20	11	16
pH, contact tank effluent	6.3	6.3	6.3	6.4	6.5	6.5	6.5	6.3	6.3	6.2	6.2	6.5	6.5	6.2	6.4
Median coliform organism concentration, MPN/100ml	2.1	3.4	3.4	2.1	<2.1	<2.1	2.1	2.1	2.1	4.0	2.1	2.1	4.0	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	7.8	9.6	5.9	11	13	11	3.3	5.4	8.5	6.1	7.7	8.1	13	3.3	8.1
Suspended solids, mg/l	33	23	42	13	39	16	9	11	14	11	19	33	42	9	22
Volatile suspended solids, mg/l	21	18	35	13	12	11	3	8	10	9	15	13	35	3	14
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	4.0	4.9	4.9	5.1	4.9	4.4	5.0	5.1	5.5	4.9	5.0	4.9	5.5	4.0	4.9
pH	6.5	6.6	6.9	6.9	6.7	7.0	7.3	7.2	7.0	6.7	6.9	6.7	7.3	6.5	6.9
Grease, mg/l	7.8	4.6	6.4	2.7	7.9	5.5	7.8	1.1	1.6	1.2	2.9	12.0	12.0	1.1	5.1
MBAS, mg/l	0.05	0.06	0.07	0.07	0.30	0.07	0.06	0.06	0.05	0.05	0.04	0.05	0.30	0.04	0.08
Turbidity, JTU	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Temperature, °C	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Ammonia nitrogen, mg/l[a]	0.9	0.2	<0.1	<0.1	<0.1	-	<0.1	0.5	0.41	0	0.43	0.44	0.9	<0.1	0.3
Nitrite nitrogen, mg/l[a]	0.02	<0.01	<0.01	<0.01	<0.01	-	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	0.02	0.01	0.01
Nitrate nitrogen, mg/l[a]	25	22	24	26	22	-	20	21	20	15	25	28	28	15	22
Organic nitrogen, mg/l[a]	3.7	3.6	4.8	2.4	1.7	-	1.7	1.7	1.6	1.6	2.7	4.0	4.8	1.6	2.7
Total solids, mg/l	800	800	810	770	680	720	810	760	710	650	720	870	870	650	760
Total dissolved solids, mg/l	[illegible]	700	770	750	650	700	830	740	690	640	700	840	840	640	740
Fixed dissolved solids, mg/l	470	440	520	490	350	400	570	550	520	340	500	540	570	340	480
Specific conductivity, micromhos	910	920	950	870	820	910	850	880	900	870	900	1,040	1,040	850	940
Chlorides, mg/l	140	160	150	150	130	140	150	200	160	150	170	200	200	130	160
Dissolved residue (by calculation), mg/l[a]	640	640	670	630	-	-	-	660	600	560	-	720	720	560	660
Alkalinity, mg/l[a]	27	46	63	50	66	-	90	77	60	34	67	48	90	27	52
Hardness, mg/l[a]	180	190	200	190	180	-	200	190	170	160	200	230	230	160	190
Boron, mg/l[a]	1.1	1.4	0.8	0.7	1.0	-	1.1	1.4	0.8	0.7	0.8	0.9	1.4	0.7	1.0
Percent sodium[a]	58	57	60	55	-	-	58	62	58	57	54	54	62	54	57
Overall Plant Removals, Percent															
BOD_5	97	96	98	96	93	96	98	98	96	97	96	96	98	93	96
Suspended solids	90	91	86	94	81	93	97	96	94	96	93	87	97	81	92
Ammonia nitrogen	98	>99	-	>99	>99	-	99	98	99	>99	99	99	>99	98	99

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

TABLE E-4F. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1973

Parameter	Month														
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	High	Low	Average
Disinfection															
Chlorine dose, mg/l	27	26	29	52	50	37	30	31	38	38	35	36	52	26	36
Contact time, min.	51	51	51	60	61	59	-	63	62	66	63	74	74	51	60
Chlorine residual, mg/l	15	14	19	18	14	17	12	13	17	19	20	22	22	12	17
pH, contact tank effluent	6.6	6.5	6.5	6.5	6.7	6.5	6.8	6.8	6.5	6.5	6.5	6.4	6.7	6.4	6.6
Median coliform organism concentration, MPN/100ml	2.1	2.1	<2.1	2.1	<2.1	2.1	2.1	6.7	<2.1	2.1	2.1	2.1	6.7	<2.1	2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	7.2	9.5	11	12	9.0	8.5	11	5.3	7.5	12	9.6	11	11	5.3	9.4
Suspended solids, mg/l	34	30	38	32	10	17	16	37	26	33	16	36	37	10	28
Volatile suspended solids, mg/l	28	23	21	18	8	13	12	8	22	15	13	29	29	8	18
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	4.9	4.2	4.1	4.6	5.0	5.0	4.6	5.3	5.1	5.3	4.5	4.7	5.3	4.1	4.8
pH	7.4	6.8	6.8	6.9	6.9	7.0	6.7	7.1	7.0	7.2	6.7	6.5	7.4	6.5	6.9
Grease, mg/l	3.7	1.1	2.5	2.8	1.6	3.0	3.5	3.8	6.7	7.2	11	14	14	1.6	5.2
MBAS, mg/l	0.05	0.04	0.05	0.05	0.03	0.04	0.30	0.04	0.03	0.03	0.03	0.07	0.30	0.03	0.06
Turbidity, JTU	7.0	12	11	14	9.3	4.4	6.2	5.7	5.6	6.5	6.4	11	14	4.4	8.3
Temperature, °C	16	17	18	20	23	25	25	25	24	22	20	17	25	16	21
Ammonia nitrogen, mg/l[a]	0.34	0.32	0.40	0.41	0.39	1.4	6.4	0.73	0.51	1.9	0.34	1.8	6.4	0.32	1.2
Nitrite nitrogen, mg/l[a]	0.003	0.002	0.002	0.002	0.003	0.002	0.009	0.003	0.002	0.003	0.001	0.002	0.009	0.001	0.002
Nitrate nitrogen, mg/l[a]	29	24	23	25	23	22	17	22	23	23	24	23	29	17	23
Organic nitrogen, mg/l[a]	2.7	5.2	4.6	6.6	2.1	4.3	2.4	1.0	1.1	0.1	0.2	0.7	6.6	0.1	2.6
Total solids, mg/l	960	950	980	960	870	810	870	840	760	750	850	770	980	750	860
Total dissolved solids, mg/l	930	920	940	940	860	790	860	800	740	720	830	730	940	720	840
Fixed dissolved solids, mg/l	560	520	670	680	600	540	590	510	480	350	450	540	680	350	540
Specific conductivity, micromhos	1,220	1,200	1,250	1,320	1,290	1,100	1,080	1,130	960	1,020	990	1,120	1,320	960	1,230
Chlorides, mg/l	220	250	250	250	210	180	170	210	180	170	190	180	250	170	210
Dissolved residue (by calculation), mg/l[a]	840	820	890	870	760	690	720	740	660	650	700	690	890	650	750
Alkalinity, mg/l[a]	66	51	51	65	61	67	80	68	50	61	56	17	80	17	58
Hardness, mg/l[a]	250	230	230	240	230	200	180	190	180	190	270	170	250	170	210
Boron, mg/l[a]	1.0	1.1	1.5	1.4	1.0	1.0	1.1	1.0	0.8	0.5	1.1	1.0	1.5	0.5	1.1
Percent sodium[a]	59	60	65	62	57	58	64	66	61	58	57	65	66	57	61
Overall Plant Removals, Percent															
BOD_5	97	96	95	95	96	96	95	98	96	94	96	95	98	94	96
Suspended solids	84	83	83	89	96	94	93	86	90	82	94	84	96	82	88
Ammonia nitrogen	-	-	99	99	99	98	-	97	98	96	-	-	99	96	98

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

TABLE E-4G. DISINFECTION, FINAL EFFLUENT CHARACTERISTICS, AND OVERALL PLANT PERFORMANCE, 1974

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Disinfection															
Chlorine dose, mg/l	35	32	34	39	42	53	52	33	34	34	32	51	53	32	39
Contact time, min.	65	72	73	69	60	56	56	57	58	60	58	-	73	56	62
Chlorine residual, mg/l	19	19	20	17	17	19	16	16	14	18	14	21	21	14	18
pH, contact tank effluent	6.8	7.3	6.6	6.4	6.5	6.4	6.5	6.7	6.4	7.0	6.6	-	7.3	6.4	6.7
Median coliform organism concentration, MPN/100ml	<2.1	2.1	2.1	2.1	<2.1	<2.1	<2.1	<2.1	<2.1	<2.1	<2.1	-	2.1	<2.1	<2.1[c]
Final Effluent Characteristics															
BOD_5, mg/l	20	18	13	9.8	11	14	9.4	5.1	11	7.8	5.9	-	20	5.1	11
Suspended solids, mg/l	38	23	23	23	16	15	16	11	10	10	15	-	38	10	18
Volatile, suspended solids, mg/l	34	14	18	17	19	11	12	11	5	20	10	-	34	5	16
Settleable solids, ml/l	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	<0.1	-	-	<0.1
Dissolved oxygen, mg/l	4.5	4.8	5.6	4.5	5.4	5.2	4.8	5.3	4.8	4.8	4.7	4.9	5.6	4.5	5.0
pH	6.8	7.3	6.6	6.4	6.5	6.4	6.5	6.7	6.4	7.0	6.6	-	7.3	6.4	6.7
Grease, mg/l	11	13	11	5.5	9.8	5.5	4.2	3.8	3.3	4.4	6.7	-	13	3.3	7.1
MBAS, mg/l	0.05	0.04	0.06	0.03	0.03	0.27	0.30	0.35	0.28	0.23	0.25	-	0.35	0.03	0.17
Turbidity, JTU	14	15	16	18	11	11	8.8	5.6	5.7	5.1	9.0	-	16	5.2	11
Temperature, °C	16	16	18	19	21	23	25	25	25	23	21	-	25	16	21
Ammonia nitrogen, mg/l[a]	0.25	5.2	0.11	0.45	0.17	0.17	1.7	<0.05	0.73	0.90	0.39	0.28	5.2	0.05	0.73
Nitrite nitrogen, mg/l[a]	0.004	0.011	0.004	0.003	0.002	0.002	<0.001	0.009	0.001	0.002	0.009	0.005	0.011	0.001	0.004
Nitrate nitrogen, mg/l[a]	30	21	26	27	25	24	22	22	23	26	26	27	30	20	25
Organic nitrogen, mg/l[a]	0.93	2.9	2.0	2.4	2.3	0.33	1.7	0.22	0.05	0.60	2.3	1.9	2.9	0.05	1.5
Total solids, mg/l	850	870	920	830	830	830	710	720	700	730	840	-	920	700	800
Total dissolved solids, mg/l	810	860	900	800	800	810	700	710	690	720	820	-	900	690	780
Fixed dissolved solids, mg/l	600	600	630	590	540	550	510	480	500	450	540	-	630	450	540
Specific conductivity, micromhos	1,100	1,090	1,100	1,230	1,170	1,000	1,020	980	960	1,030	980	-	1,230	960	1,060
Chlorides, mg/l	200	210	210	210	200	190	170	160	170	160	160	-	210	160	190
Dissolved residue (by calculation), mg/l[a]	770	770	770	770	730	670	680	660	620	620	-	-	770	620	710
Alkalinity, mg/l[a]	20	62	15	16	31	18	42	26	38	29	62	29	62	15	32
Hardness, mg/l[a]	190	200	210	200	190	180	160	180	170	180	230	220	230	160	190
Boron, mg/l[a]	0.88	1.1	1.1	1.1	0.95	0.96	0.91	0.8	0.6	0.9	0.9	0.8	1.1	0.6	0.9
Percent sodium[a]	62	60	61	60	61	61	60	60	59	56	53	56	62	53	59
Overall Plant Removals, Percent															
BOD_5	91	92	95	95	95	94	95	98	95	96	97	-	98	91	95
Suspended solids	87	90	90	92	92	93	93	96	95	95	95	-	96	87	93
Ammonia nitrogen	-	-	-	-	-	99	-	-	98	98	99	-	99	98	98

[a] Analyses performed by Environmental Quality Analysts, Inc., San Francisco

[b] Based on primary effluent characteristics

[c] Median

TABLE E-5A. SOLIDS HANDLING AND TREATMENT, 1968

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	22,000	17,200	16,800	17,400	17,100	16,400	14,900	15,100	18,500	17,100	16,000	16,800	22,000	14,900	17,100
To lagoons	20,300	17,800	18,000	20,000	24,400	23,000	18,500	14,200	18,300	15,900	13,300	17,100	24,400	13,300	18,400
Digester Performance															
Total solids, percent															
Raw[b]	3.2	3.7	4.1	3.7	3.8	3.6	3.5	3.9	3.6	4.3	4.1	4.2	4.3	3.2	3.8
Digested															
No. 1	1.6	1.4	1.9	1.0	1.3	1.0	2.4	1.6	1.9	2.1	2.6	2.1	2.6	1.0	1.8
No. 2	2.0	1.9	2.7	1.9	2.0	2.4	2.3	2.1	2.1	2.6	3.1	3.7	3.7	1.9	2.3
Volatile solids, percent of total															
Raw[b]	81	80	78	80	80	82	80	83	80	82	73	79	83	73	80
Digested															
No. 1	74	67	68	66	66	64	67	68	64	64	63	68	74	63	67
No. 2	74	69	67	65	67	69	63	70	68	68	64	70	74	63	68
Volatile solids reduction, percent															
No. 1	33	49	40	51	51	59	49	56	54	61	37	44	61	33	49
No. 2	33	44	43	54	49	51	57	52	47	53	34	38	57	33	46
Digester temperature, F															
No. 1	70	81	88	93	93	97	97	96	96	95	95	88	97	70	91
No. 2	77	80	85	89	90	91	97	97	93	95	92	87	97	77	89
Digester gas produced, 1,000 ft^3/day[c,d]															
No. 1	11.1	22.9	25.2	25.4	26.1	26.3	23.8	22.9	19.8	18.1	17.4	20.9	26.3	11.1	21.7
No. 2	2.6	9.1	10.2	8.6	9.5	8.1	7.0	6.0	3.2	2.3	7.6	8.9	10.2	2.3	6.9
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	470	260	340	150	130	150	420	180	140	280	140	560	560	130	270
No. 2	420	320	270	200	180	400	240	250	200	1,280	510	350	1,280	180	390
Alkalinity, mg/l															
No. 1	1,740	1,910	2,020	1,840	1,880	1,880	1,680	1,550	1,480	1,840	1,900	1,850	2,020	1,480	1,800
No. 2	2,970	2,430	2,660	2,620	2,740	2,480	2,710	2,400	2,380	2,320	2,900	2,850	2,900	2,070	2,550
pH															
No. 1	6.6	6.8	6.9	7.0	7.0	7.0	6.7	6.8	6.8	6.8	6.9	6.7	7.0	6.6	6.8
No. 2	6.8	6.8	7.0	7.0	7.0	7.0	6.9	7.0	6.8	6.4	7.0	6.8	7.0	6.4	6.8

[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

[d] Digester gas contained an average of 36 percent CO_2 in 1968.

TABLE E-5B. SOLIDS HANDLING AND TREATMENT, 1969

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	19,000	20,700	18,200	16,800	17,100	18,900	16,700	15,900	17,700	17,700	17,700	19,600	20,700	15,900	18,000
To lagoons	18,000	19,900	18,100	16,600	17,900	15,500	16,200	16,600	16,100	18,300	17,100	20,100	20,100	15,500	17,500
Digester Performance															
Total solids, percent															
Raw[b]	4.5	4.8	4.6	4.1	3.4	3.5	3.2	3.0	3.1	3.8	3.6	3.9	4.8	3.0	3.8
Digested															
No. 1	3.7	1.8	4.1	1.5	2.7	2.1	1.7	2.1	2.1	2.4	1.6	1.7	4.1	1.5	2.2
No. 2	3.1	2.9	2.9	1.6	1.7	2.5	2.0	1.9	1.5	2.0	1.6	1.4	3.1	1.4	2.1
Volatile solids, percent of total															
Raw[b]	71	73	83	84	85	83	84	80	86	85	85	86	86	71	82
Digested															
No. 1	56	58	61	64	72	72	70	61	72	73	70	72	73	56	67
No. 2	67	57	60	64	72	69	71	63	73	75	71	71	75	57	68
Volatile solids reduction, percent															
No. 1	47	49	68	66	65	47	56	61	58	52	59	58	68	47	56
No. 2	17	51	71	66	55	54	53	57	56	47	57	60	71	17	54
Digester temperature, F															
No. 1	84	80	83	87	94	97	98	91	87	92	97	92	98	80	90
No. 2	81	81	84	91	93	94	92	93	97	94	96	90	97	81	91
Digester gas produced, 1,000 ft³/day[c]															
No. 1	18.1	16.8	10.6	16.4	15.1	17.5	-	-	-	-	-	-	18.1	10.6	15.3
No. 2	8.0	10.3	16.3	10.9	9.8	10.3	9.6	11.4	11.4	9.6	15.4	13.3	16.3	8.0	11.4
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	200	380	220	230	280	230	200	220	250	440	280	240	440	200	260
No. 2	550	310	260	200	200	380	240	240	210	650	310	280	650	200	320
Alkalinity, mg/l															
No. 1	1,920	1,810	2,060	2,600	3,030	2,730	2,600	2,420	2,560	2,830	3,000	2,840	3,030	1,810	2,540
No. 2	2,740	2,740	3,040	2,750	3,000	2,760	2,470	2,360	2,620	2,470	2,590	2,400	3,040	2,360	2,660
pH															
No. 1	6.9	6.7	6.7	7.0	7.1	7.1	7.1	7.0	6.8	6.9	7.1	6.9	7.1	6.7	6.9
No. 2	6.9	6.9	7.1	7.0	7.1	7.0	7.0	7.0	6.9	6.8	7.0	6.9	7.1	6.8	7.0

[a] gpd x 0.0038 = m³/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft³ x 0.0283 = 1,000 m³

TABLE E-5C. SOLIDS HANDLING AND TREATMENT, 1970

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	19,500	19,300	19,500	19,700	18,400	18,300	18,900	18,300	18,100	18,700	18,600	18,800	19,700	18,100	18,800
To lagoons	19,500	18,800	19,600	19,200	18,500	18,000	18,400	18,600	18,200	18,200	18,700	18,300	19,600	18,000	18,700
Digester Performance															
Total solids, percent															
Raw[b]	3.4	2.8	3.5	3.4	3.6	3.5	3.5	3.9	3.4	3.8	3.4	4.1	4.1	2.8	3.5
Digested															
No. 1	2.0	1.5	1.4	1.6	1.6	1.5	1.7	1.7	1.6	1.8	1.6	2.2	2.2	1.4	1.7
No. 2	1.6	0.9	1.2	1.5	1.7	1.7	1.4	1.8	1.6	1.7	1.6	1.8	1.8	0.9	1.5
Volatile solids, percent of total															
Raw[b]	83	88	83	85	84	84	84	85	83	84	82	83	88	82	84
Digested															
No. 1	75	69	70	69	70	69	71	71	73	73	71	71	75	69	71
No. 2	72	69	69	72	68	70	71	72	74	73	71	68	74	68	71
Volatile solids reduction, percent															
No. 1	39	70	52	61	56	58	53	57	45	49	46	50	70	39	53
No. 2	47	70	54	55	60	56	53	55	42	49	46	57	70	42	54
Digester temperature, F															
No. 1	84	85	91	90	89	94	98	97	96	92	91	82	98	82	91
No. 2	84	85	95	88	94	94	97	96	96	92	94	83	97	83	92
Digester gas produced, 1,000 ft^3/day[c]															
No. 1	-	10.9	14.2	6.6	20.3	18.5	13.7	13.2	12.6	15.3	16.2	14.1	20.3	6.6	14.1
No. 2	9.7	13.7	16.3	12.3	11.1	-	-	-	-	-	-	-	16.3	9.7	12.6
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	320	240	300	1,750	510	190	600	730	820	360	280	370	1,750	190	540
No. 2	690	220	200	1,030	1,270	640	700	1,020	560	370	360	510	1,270	200	630
Alkalinity, mg/l															
No. 1	2,550	2,440	2,740	3,440	2,900	2,610	2,500	2,460	2,500	2,360	2,450	2,610	3,440	2,360	2,630
No. 2	2,240	1,710	2,020	2,740	3,380	2,470	2,270	2,400	2,350	2,540	2,420	2,480	3,380	1,710	2,420
pH															
No. 1	7.0	7.0	7.0	6.7	7.0	7.0	7.0	6.8	6.9	7.0	6.8	6.9	7.0	6.7	6.9
No. 2	6.8	6.8	6.9	6.7	7.1	7.0	6.9	6.8	7.0	7.0	6.8	6.8	7.1	6.7	6.9

[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-5D. SOLIDS HANDLING AND TREATMENT, 1971

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	20,700	20,700	21,000	20,700	20,700	21,600	21,300	20,500	18,900	18,700	20,700	21,200	21,600	18,700	20,600
To lagoons	19,800	20,600	20,500	20,800	20,300	21,600	21,000	20,200	18,700	18,700	20,500	20,800	21,600	18,700	20,300
Digester Performance															
Total solids, percent															
Raw[b]	3.8	3.1	2.8	3.6	3.2	3.5	3.4	3.6	3.8	3.3	3.8	3.7	3.8	2.8	3.5
Digested															
No. 1	2.0	1.4	1.4	1.5	1.7	1.4	1.6	1.8	1.6	1.4	1.7	1.7	2.0	1.4	1.6
No. 2	2.1	1.3	1.4	1.7	1.6	1.5	1.7	1.5	2.0	1.4	1.7	1.6	2.1	1.3	1.6
Volatile solids, percent of total															
Raw[b]	87	84	83	82	83	85	84	83	85	84	85	84	87	82	84
Digested															
No. 1	73	73	73	69	72	70	68	71	69	70	73	70	73	68	71
No. 2	73	71	73	70	72	71	70	70	69	70	74	70	74	69	71
Volatile solids reduction, percent															
No. 1	60	49	45	51	47	59	60	50	61	56	52	56	61	45	54
No. 2	60	53	45	50	47	57	56	52	61	56	50	56	61	45	54
Digester temperature, F															
No. 1	78	84	92	97	95	95	99	98	98	96	91	84	99	78	92
No. 2	85	95	95	98	98	98	99	100	99	98	92	93	100	85	96
Digester gas produced, 1,000 ft^3/day[c]															
No. 1	14.4	12.4	13.5	14.9	15.4	14.4	15.0	18.5	13.1	7.9	11.9	10.6	15.4	7.9	12.7
No. 2	-	-	-	19.5	20.0	19.5	18.0	16.3	18.7	17.5	16.2	20.3	20.3	16.2	18.4
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	380	280	240	280	210	250	310	1,260	350	600	510	360	1,260	210	420
No. 2	370	300	240	260	290	260	250	800	610	340	570	340	800	240	390
Alkalinity, mg/l															
No. 1	2,510	2,380	2,500	2,460	2,650	2,010	2,020	2,390	2,620	2,020	2,330	2,140	2,650	2,010	2,340
No. 2	2,440	2,420	2,440	2,420	2,340	2,180	2,300	2,350	2,650	2,280	2,370	2,160	2,650	2,160	2,360
pH															
No. 1	6.8	6.9	7.0	6.9	6.9	6.8	6.9	6.5	6.8	6.5	6.4	6.6	7.0	6.4	6.8
No. 2	6.8	6.9	7.0	6.9	6.8	6.8	6.9	6.6	6.8	6.6	6.4	6.7	7.0	6.4	6.8

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[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-5E. SOLIDS HANDLING AND TREATMENT, 1972

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	22,200	22,700	21,600	21,600	22,700	20,900	20,900	19,300	21,400	27,600	23,300	24,000	27,600	19,300	22,300
To lagoons	21,000	21,500	21,500	21,200	21,600	21,100	20,400	19,300	21,600	27,400	22,900	23,700	27,400	19,300	21,900
Digester Performance															
Total solids, percent															
Raw[b]	-	4.3	4.0	3.6	4.0	-	4.5	-	3.5	3.1	3.4	3.0	4.5	3.0	3.7
Digested															
No. 1	-	1.4	1.9	1.9	2.8	-	1.5	-	1.6	2.2	2.7	1.7	2.8	1.4	2.0
No. 2	-	1.5	1.7	1.5	1.2	-	1.3	-	1.3	2.0	2.8	-	2.8	1.3	1.7
Volatile solids, percent of total															
Raw[b]	-	85	78	81	86	-	83	-	83	87	83	84	87	78	83
Digested															
No. 1	-	71	68	69	72	-	67	-	73	68	71	74	74	67	70
No. 2	-	71	70	68	71	-	70	-	72	72	72	-	72	68	71
Volatile solids reduction, percent															
No. 1	-	57	40	48	58	-	58	-	45	68	50	46	68	40	52
No. 2	-	57	34	50	60	-	52	-	47	62	47	-	62	34	51
Digester temperature, F															
No. 1	80	82	90	92	94	99	97	95	98	95	93	81	99	80	91
No. 2	91	93	95	95	96	98	96	94	96	93	95	81	98	81	94
Digester gas produced, 1,000 ft³/day[c]															
No. 1	13.7	15.7	16.5	10.9	17.5	14.8	16.2	15.4	14.7	7.5	15.1	12.0	17.5	7.5	14.2
No. 2	20.3	20.4	18.1	18.1	15.6	12.5	19.2	18.2	17.0	11.2	20.3	3.4	20.4	3.4	16.4
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	280	300	260	850	500	710	200	-	540	2,430	960	310	2,430	200	670
No. 2	220	360	250	240	530	1,620	280	-	490	2,490	1,520	1,000	2,490	220	820
Alkalinity, mg/l															
No. 1	2,080	2,020	2,220	1,830	2,480	1,800	2,260	-	2,260	3,760	3,180	2,170	3,760	1,800	2,370
No. 2	2,320	2,200	2,150	2,010	1,880	2,210	2,210	-	2,080	3,470	2,950	2,140	3,470	1,880	2,330
pH															
No. 1	6.8	6.5	6.9	6.4	6.7	6.7	6.8	-	6.8	6.6	7.0	6.9	7.0	6.4	6.7
No. 2	6.9	6.7	6.8	6.7	6.4	6.3	6.7	-	6.9	6.5	6.7	6.7	6.9	6.3	6.7

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[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-5F. SOLIDS HANDLING AND TREATMENT, 1973

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	24,100	23,100	23,700	22,500	23,200	22,900	21,000	22,800	21,100	22,600	23,300	22,900	24,100	21,000	22,800
To lagoons	23,600	22,500	23,900	22,500	22,900	23,100	20,700	23,000	20,500	22,600	23,300	22,900	23,900	20,500	22,600
Digester Performance															
Total solids, percent															
Raw[b]	3.7	3.0	2.9	3.1	3.1	3.2	2.7	4.1	3.3	3.0	3.5	3.4	4.1	2.7	3.3
Digested															
No. 1	2.2	1.5	1.5	1.4	1.5	1.5	1.3	1.5	1.5	1.9	2.3	2.1	2.3	1.3	1.7
No. 2	2.2	1.0	1.3	1.2	1.2	1.7	0.8	1.2	1.2	1.6	1.7	1.9	2.2	0.8	1.4
Volatile solids, percent of total															
Raw[b]	81	86	83	83	81	81	84	83	83	84	83	86	86	81	83
Digested															
No. 1	67	66	69	68	67	68	69	69	70	71	71	72	76	66	69
No. 2	67	59	67	68	69	67	70	68	70	69	73	72	73	59	68
Volatile solids reduction, percent															
No. 1	46	68	54	56	46	50	58	54	52	53	50	58	68	46	54
No. 2	46	77	58	56	48	46	56	54	52	58	45	58	77	45	55
Digester temperature, F															
No. 1	79	79	82	86	88	93	95	91	94	92	87	87	95	79	88
No. 2	86	89	92	94	98	100	100	95	99	99	93	91	100	86	95
Digester gas produced, 1,000 ft^3/day[c]															
No. 1	12.5	13.2	13.3	7.7	10.1	10.3	10.6	10.2	9.8	7.8	4.4	3.3	13.3	3.3	9.4
No. 2	39.7	40.3	41.0	23.0	18.6	32.2	34.8	33.5	34.6	33.5	31.5	29.2	41.0	18.6	22.7
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	400	510	370	220	420	280	280	280	260	240	570	440	570	220	360
No. 2	810	240	180	180	320	230	200	310	300	250	390	320	810	180	310
Alkalinity, mg/l															
No. 1	2,270	1,760	2,120	2,130	2,060	2,240	2,400	2,240	2,410	2,400	2,200	2,580	2,580	1,760	2,240
No. 2	2,320	2,170	2,260	2,120	2,280	2,300	2,180	1,980	2,340	2,470	2,330	2,740	2,740	1,980	2,290
pH															
No. 1	6.9	6.7	6.9	6.9	6.8	6.9	7.0	6.8	6.9	6.9	6.9	7.0	7.0	6.7	6.9
No. 2	7.0	6.9	7.0	6.9	6.9	7.0	7.0	6.8	6.8	6.9	6.9	7.1	7.1	6.8	6.9

[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-5G. SOLIDS HANDLING AND TREATMENT, 1974

Parameter	Month												High	Low	Average
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Sludge Flow, gpd[a]															
To digesters[b]	23,300	22,600	20,300	21,700	22,800	21,200	21,000	21,400	20,200	21,800	21,400	19,600	23,300	19,600	21,400
To lagoons	21,200	22,500	20,200	21,900	22,800	20,800	21,500	21,500	20,000	22,300	20,300	19,400	22,800	19,400	21,200
Digester Performance															
Total solids, percent															
Raw[b]	3.1	3.2	3.1	3.0	2.9	3.1	3.1	3.9	3.3	3.5	3.4	3.4	3.9	2.9	3.3
Digested															
No. 1	1.5	1.4	-	1.3	1.2	2.0	-	2.1	1.6	-	1.8	-	2.1	1.2	1.6
No. 2	1.5	1.1	-	1.0	1.2	4.0	-	4.8	2.0	-	3.1	-	4.8	1.0	2.3
Volatile solids, percent of total															
Raw[b]	86	86	-	84	85	82	-	84	83	-	83	-	86	82	84
Digested															
No. 1	70	72	-	70	72	68	-	72	73	-	72	-	73	68	71
No. 2	73	67	-	69	71	75	-	75	70	-	72	-	75	67	72
Volatile solids reduction, percent															
No. 1	62	50	-	56	55	53	-	51	45	-	47	-	62	45	53
No. 2	56	67	-	50	57	34	-	43	52	-	47	-	67	34	52
Digester temperature, F															
No. 1	90	87	87	95	96	99	99	100	102	106	104	92	106	87	96
No. 2	85	85	88	94	101	102	107	103	103	107	96	90	107	85	97
Digester gas produced, 1,000 ft^3/day[c]															
No. 1	2.1	0	5.2	12.3	11.0	15.5	19.2	18.3	14.7	11.9	13.4	10.0	19.2	0	11.1
No. 2	28.9	25.5	19.9	19.2	17.4	10.1	10.3	10.6	13.9	17.6	13.5	14.1	28.9	10.1	16.8
Digested Sludge Characteristics															
Volatile acids, mg/l															
No. 1	490	580	360	240	260	400	260	280	620	1,160	580	-	1,160	240	480
No. 2	850	500	330	270	220	2,160	2,940	3,180	1,010	810	1,930	-	3,180	220	1,360
Alkalinity, mg/l															
No. 1	2,500	2,320	2,200	2,140	2,730	4,020	3,810	3,440	2,460	2,700	2,500	-	4,020	2,140	2,810
No. 2	2,630	2,440	2,170	2,110	2,950	5,260	4,340	3,250	3,410	2,980	2,490	-	5,260	2,110	3,080
pH															
No. 1	6.9	6.9	6.8	6.8	6.9	6.9	6.9	6.9	6.8	6.7	6.9	-	6.9	6.7	6.9
No. 2	6.7	7.0	6.8	6.6	6.9	6.0	5.4	5.4	7.1	6.9	6.3	-	7.1	5.4	6.5

[a] gpd x 0.0038 = m^3/day

[b] From primary sedimentation tanks; includes waste mixed liquor recycled to headworks

[c] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-6A. POWER, CHEMICALS, AND LABOR, 1968

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	16.5	17.4	17.8	21.5	13.6	15.3	16.5	16.2	20.5	19.7	22.9	20.1	22.9	13.6	218
Electric power, 1,000 kwh	140	134	169	163	179	183	177	215	247	258	249	250	258	134	2,360
Natural gas, 1,000 ft³[b]	229	72.0	141	148	42.8	139	104	84.5	114	91.8	98.1	104	229	72.0	1,370
Labor, Manhours															
Operation	646	608	607	548	648	639	-	350	-	648	660	654	660	350	7,210[c]
Routine maintenance	259	246	222	255	188	149	-	320	-	188	91	170	320	91	2,506[c]
Repair	293	136	165	172	168	174	-	197	-	194	295	248	295	136	2,450[c]
Laboratory and monitoring	176	262	224	199	211	252	-	286	-	294	220	210	294	176	2,801[c]
Yard	136	160	155	172	168	157	-	187	-	185	127	90	187	90	1,844[c]
Total	1,510	1,412	1,373	1,346	1,383	1,371	-	1,340	-	1,509	1,393	1,372	1,510	1,340	16,811[c]

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[a] 1,000 lb × 0.45 = 1,000 kg

[b] 1,000 ft³ × 0.0283 = 1,000 m³

[c] Adjusted for a 12-month period

TABLE E-6B. POWER, CHEMICALS, AND LABOR, 1969

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	20.1	21.7	24.2	20.9	16.4	20.3	18.2	18.4	20.8	20.6	18.2	22.6	24.2	16.4	222
Electric power, 1,000 kwh	208	215	230	230	243	230	219	215	208	210	192	224	243	192	2,620
Natural gas, 1,000 ft^3[b]	120	49.3	37.4	29.5	28.4	162	77.3	70.8	103	148[a]	39.1	87.7	162	28.4	952
Labor, Manhours															
Operation	691	588	651	630	651	592	651	651	667	651	645	737	737	588	7,805
Routine maintenance	208	213	174	200	264	103	250	118	40	277	192	121	277	40	2,160
Repair	242	136	120	231	368	236	195	212	266	259	219	371	368	120	2,855
Laboratory and monitoring	286	318	281	277	249	228	251	305	236	269	232	202	318	232	3,214
Yard	98	120	147	105	164	136	16	168	158	188	124	81	188	16	1,505
Part-time, unclassified	-	-	-	-	-	264	538	380	-	-	-	-	538	264	1,182
Total	1,525	1,375	1,373	1,523	1,696	1,559	1,901	1,834	1,367	1,644	1,412	1,592	1,834	1,367	18,801

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[a] 1,000 lb x 0.45 = 1,000 kg

[b] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-6C. POWER, CHEMICALS, AND LABOR, 1970

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	27.0	23.3	26.2	23.2	23.7	24.1	23.9	20.1	24.7	28.9	25.3	24.9	28.9	20.1	295
Electric power, 1,000 kwh	242	220	249	233	232	228	230	240	209	208	204	270	270	204	2,770
Natural gas, 1,000 ft^3 [b]	50.1	35.0	45.9	87.2	61.5	16.1	17.0	33.8	20.0	90.0	7.3	7.9	90.0	7.3	501
Labor, Manhours															
Operation	713	648	707	682	690	730	735	739	730	769	728	735	769	648	8,605
Routine maintenance	138	170	224	242	101	344	622	426	193	54	139	180	622	54	2,923
Repair	267	344	236	208	319	331	315	229	177	198	101	75	344	75	2,800
Laboratory and monitoring	343	260	299	282	284	284	270	229	262	270	231	258	343	229	3,272
Yard	94	101	164	173	150	176	213	322	160	174	72	109	322	72	1,916
Total	1,555	1,523	1,630	1,597	1,643	1,865	2,155	1,945	1,530	1,445	1,271	1,357	2,155	1,271	19,516

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[a] 1,000 lb x 0.45 = 1,000 kg

[b] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-6D. POWER, CHEMICALS, AND LABOR, 1971

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	35.1	32.2	22.3	25.6	32.5	33.4	36.0	34.5	29.0	32.9	37.7	39.1	39.1	22.3	390
Electric power, 1,000 kwh	211	193	199	214	209	212	227	230	230	236	230	230	236	193	2,620
Natural gas, 1,000 ft^3[b]	8.1	7.3	8.4	6.6	3.9	3.4	3.7	12.6	13.3	14.1	13.4	13.9	14.1	3.4	109
Labor, Manhours															
Operation	736	677	763	720	744	720	744	744	720	744	744	744	763	677	8,820
Routine maintenance	257	214	-	182	202	229	290	138	369	135	193	144	369	135	2,567[c]
Repair	290	172	-	283	195	196	244	166	162	183	240	252	290	162	2,630[c]
Laboratory and monitoring	250	234	-	252	241	246	263	217	239	240	237	251	263	217	2,913[c]
Yard	121	147	132	165	152	120	192	162	142	144	71	52	192	52	1,600
Total	1,654	1,444	-	1,602	1,534	1,511	1,733	1,447	1,632	1,446	1,493	1,443	1,733	1,439	18,530[c]

[a] 1,000 lb x 0.45 = 1,000 kg

[b] 1,000 ft^3 x 0.0283 = 1,000 m^3

[c] Adjusted for a 12-month period

TABLE E-6E. POWER, CHEMICALS, AND LABOR, 1972

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	36.7	35.8	39.2	39.6	42.4	34.5	35.0	33.7	33.1	40.2	42.0	47.0	47.0	33.1	459
Electric power, 1,000 kwh	225	243	250	261	252	247	251	244	230	263	292	293	293	225	3,040
Natural gas, 1,000 ft^3[b]	14.0	12.9	14.5	13.2	16.7	13.1	13.0	12.9	12.5	104	14.6	14.5	104	12.5	254
Labor, Manhours															
Operation	768	672	844	720	744	720	744	794	720	744	700	816	844	672	9,062
Routine maintenance	188	240	184	160	168	336	184	212	195	97	242	141	504	97	2,748
Repair	176	196	244	344	201	244	205	272	154	176	232	263	344	176	2,695
Laboratory and monitoring	188	239	280	274	250	202	152	140	156	267	176	256	280	140	2,570
Yard	59	152	170	162	167	195	168	154	158	157	152	40	195	40	1,762
Total	1,356	1,497	1,710	1,660	1,530	1,697	1,793	1,672	1,393	1,421	1,502	1,518	1,793	1,356	18,837

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[a] 1,000 lb × 0.45 = 1,000 kg

[b] 1,000 ft^3 × 0.0283 = 1,000 m^3

TABLE E-6F. POWER, CHEMICALS, AND LABOR, 1973

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	41.3	36.3	45.5	42.8	41.7	31.7	33.9	34.2	39.9	42.0	40.3	41.9	45.5	31.7	472
Electric power, 1,000 kwh	294	262	289	278	281	272	289	291	266	274	272	285	294	262	3,350
Natural gas, 1,000 ft^3 [b]	14.3	68.6	14.0	12.7	13.4	13.3	13.5	13.3	12.7	14.2	13.8	15.0	68.6	12.7	219
Labor, Manhours															
Operation	819	732	840	706	824	710	814	809	690	728	664	814	840	664	9,150
Routine maintenance	199	156	240	168	140	387	504	315	151	276	299	162	504	140	2,997
Repair	262	170	210	246	258	247	263	266	292	260	240	241	292	170	2,963
Laboratory and monitoring	259	228	247	209	269	243	247	232	242	259	243	217	269	209	2,895
Yard	89	63	128	160	157	216	144	264	82	65	113	63	264	63	1,544
Total	1,628	1,349	1,665	1,489	1,648	1,803	1,972	1,886	1,457	1,588	1,567	1,497	1,972	1,349	19,549

[a] 1,000 lb x 0.45 = 1,000 kg

[b] 1,000 ft^3 x 0.0283 = 1,000 m^3

TABLE E-6G. POWER, CHEMICALS, AND LABOR, 1974

Parameter	Month												High	Low	Total
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec			
Power and Chemicals															
Chlorine, 1,000 lb[a]	40.0	34.1	42.4	49.0	50.7	54.1	51.6	43.8	45.8	48.7	43.7	-	54.1	34.1	549
Electric power, 1,000 kwh	272	195	280	266	262	252	267	268	274	277	267	-	280	195	3,141
Natural gas, 1,000 ft^3 [b]	15.1	13.8	15.1	14.3	14.2	15.1	15.1	14.9	14.4	14.9	14.4	-	15.1	13.8	176
Labor, Manhours															
Operation	729	648	731	714	738	701	822	720	744	743	698	-	822	648	8,714[c]
Routine maintenance	179	206	208	268	275	399	506	453	266	390	329	-	506	179	3,795[c]
Repair	288	252	361	313	312	316	306	341	310	334	272	-	361	252	3,714[c]
Laboratory and monitoring	271	208	266	255	246	219	263	271	230	287	238	-	287	208	3,004[c]
Yard	96	80	80	173	176	160	160	187	160	160	64	-	187	80	1,632[c]
Total	1,563	1,394	1,646	1,723	1,747	1,795	2,057	1,972	1,740	1,914	1,601	-	2,057	1,394	20,859[c]

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[a] 1,000 lb x 0.45 = 1,000 kg

[b] 1,000 ft^3 x 0.0283 = 1,000 m^3

[c] Adjusted for a 12-month period

TABLE E-7A. FINAL EFFLUENT HEAVY METALS, 1968

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	<0.01	-	<0.01	-	<0.01
Barium	<0.1	-	<0.1	-	<0.1
Cadmium	<0.01	-	<0.01	-	<0.01
Hexavalent Cr	<0.003	-	<0.005	-	<0.004
Cyanide	<0.001	-	0.017	-	0.0014
Lead	<0.03	-	<0.03	-	<0.03
Selenium	0.006	-	<0.001	-	0.0035
Silver	0.013	-	<0.02	-	0.016

[a] 6-mo composites used in 1968-71

TABLE E-7B. FINAL EFFLUENT HEAVY METALS, 1969

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	0.001	-	0.001	-	0.001
Barium	0.1	-	0.3	-	0.20
Cadmium	0.004	-	0.003	-	0.0015
Hexavalent Cr	0.030	-	0.010	-	0.015
Cyanide	0.012	-	0.004	-	0.008
Lead	0.03	-	0.04	-	0.035
Selenium	0.008	-	0.003	-	0.0055
Silver	0.01	-	0.05	-	0.030

[a] 6-mo composites used in 1968-71

TABLE E-7C. FINAL EFFLUENT HEAVY METALS, 1970

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	0.002	-	0.001	-	0.0015
Barium	<0.05	-	<0.05	-	<0.05
Cadmium	<0.01	-	<0.01	-	<0.01
Hexavalent Cr	<0.01	-	<0.01	-	<0.01
Cyanide	0.019	-	0.016	-	0.018
Lead	0.013	-	<0.01	-	0.07
Selenium	<0.001	-	0.001	-	<0.001
Silver	<0.01	-	<0.05	-	<0.03

[a] 6-mo composites used in 1968-71

TABLE E-7D. FINAL EFFLUENT HEAVY METALS, 1971

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	0.0013	-	0.001	-	0.0012
Barium	<0.05	-	<0.1	-	<0.1
Cadmium	<0.01	-	0.002	-	<0.01
Hexavalent Cr	<0.01	-	<0.01	-	<0.01
Cyanide	0.07	-	0.017	-	0.044
Lead	0.13	-	0.01	-	0.06
Selenium	0.0015	-	0.003	-	0.0023
Silver	0.01	-	<0.02	-	0.01

[a] 6-mo composites used in 1968-71

TABLE E-7E. FINAL EFFLUENT HEAVY METALS, 1972

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	<0.001	0.003	0.001	0.003	0.002
Barium	<0.001	<0.1	<0.1	<0.1	<0.075
Cadmium	<0.001	<0.1	0.005	0.005	0.008
Hexavalent Cr	<0.01	<0.01	0.005	0.005	0.008
Cyanide	0.025	0.009	0.030	0.024	0.022
Lead	0.025	0.011	0.097	0.037	0.043
Selenium	0.001	0.006	0.002	<0.001	0.003
Silver	<0.02	0.01	<0.01	<0.002	0.011

[a] 3-mo composites used in 1972-74

TABLE E-7F. FINAL EFFLUENT HEAVY METALS, 1973

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	0.001	0.002	0.001	0.002	0.001
Barium	<0.1	<0.1	<0.1	<0.1	<0.1
Cadmium	0.001	0.002	0.002	0.001	0.001
Hexavalent Cr	0.001	0.002	0.001	0.004	0.002
Cyanide	0.038	0.074	0.050	0.055	0.054
Lead	0.030	0.015	0.030	0.014	0.022
Selenium	0.014	0.004	<0.001	0.004	0.005
Silver	0.005	0.007	0.005	0.002	0.004

[a] 3-mo composites used in 1972-74

TABLE E-7G. FINAL EFFLUENT HEAVY METALS, 1974

Constituent	Concentration, mg/l[a]				
	Mar	June	Sept	Dec	Avg.
Arsenic	0.002	0.002	<0.001	0.004	0.002
Barium	<0.1	<0.1	<0.1	<0.1	<0.1
Cadmium	0.002	0.002	0.001	<0.001	0.001
Hexavalent Cr	0.004	0.004	0.015	0.004	0.006
Cyanide	0.097	0.070	0.029	0.003	0.049
Lead	0.012	0.010	0.003	0.004	0.007
Selenium	0.003	0.001	<0.001	<0.001	0.001
Silver	0.007	0.005	0.004	0.004	0.005

[a] 3-mo composites used in 1972-74